STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF RESOURCES PLANNING

BULLETIN No. 63

SEA-WATER INTRUSION IN CALIFORNIA

APPENDIX C

PART I—LABORATORY AND MODEL STUDIES

PART II—AN ABSTRACT OF LITERATURE

PART III—REVIEW OF FORMULAS AND DERIVATIONS

APPENDIX D

AN INVESTIGATION OF SOME PROBLEMS IN PREVENTING SEA-WATER INTRUSION BY CREATING A FRESH-WATER BARRIER

APPENDIX E

PRELIMINARY CHEMICAL-QUALITY STUDY IN THE MANHATTAN BEACH AREA, CALIFORNIA

EDMUND G. BROWN
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FOREWORD

Supporting data for Bulletin No. 63 "Sea-Water Intrusion in California" are contained in five appendixes. Detailed information regarding geology, hydrology, and water quality in the 262 ground water basins included in the investigations are tabulated in Appendix A, published as a separate volume. A report by the Los Angeles County Flood Control District on the West Coast Basin Experimental Project is published separately as Appendix B. The remaining three appendixes, included in this volume, present reports on the following specialized studies:

Appendix C, Part I	--	"Laboratory and Model Studies of Sea-Water Intrusion"
Appendix C, Part II	--	"An Abstract of Literature Pertaining to Sea-Water Intrusion and its Control"
Appendix C, Part III	--	"Review of Formulas and Derivations for the Equilibrium Rate of Seaward Flow in a Coastal Aquifer with Sea-Water Intrusion"
Appendix D	--	"An Investigation of Some Problems in Preventing Sea-Water Intrusion By Creating a Fresh-Water Barrier"
Appendix E	--	"Preliminary Chemical-Quality Study in the Manhattan Beach Area, California"

Except for minor editorial treatment, these reports are presented substantially as submitted by the authors. Some of these reports have been published previously by the originating agencies. Appropriate notations are included in the individual title sheets to identify these publications.

APPENDIX C, PART I

LABORATORY AND MODEL STUDIES OF SEA-WATER INTRUSION

Sanitary Engineering Research Laboratory
University of California, Berkeley

Published as Technical Bulletin 11, I.E.R.
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REPORT ON

LABORATORY AND MODEL STUDIES

of

SEA WATER INTRUSION

Sanitary Engineering Research Laboratory
Department of Engineering

University of California
Berkeley

May 1955

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* Took part in investigation but left research staff at close of project.

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PREFACE

Need for Study of Sea Water Intrusion

Since about the beginning of World War II the rate of development and exploitation of California's ground water resources has proceeded at an accelerated pace in scale with the state's expanding economy and population. This has resulted in the creation of serious problems of overdraft, with extensive damage to water resources by sea water intrusion into at least thirteen ground water basins bordering the coast and inland bays of California. Further deterioration is inevitable in these areas and a number of other basins will likewise suffer unless remedial steps are taken. Ultimately, of course, withdrawals from any ground water basin must be limited by the rate of natural and artificial recharge. Profound legal, engineering, economic and political considerations, however, are involved in the use of existing ground waters, and in the provision of water from alternate sources in sufficient quantities to offset the present overdraft and to make possible continued economic growth of the area. Even if it were feasible summarily to halt all ground water withdrawals in some of the damaged basins until natural recharge restored a favorable water table, the encroachment of sea water could be expected to continue to extend itself during the considerable period of time required for such restoration to be accomplished.

In view of the foregoing considerations, proposals were made for the creation of fresh water mounds by direct recharge of reclaimed waters, or the construction of relatively impervious membranes near the ocean front, for the purpose of limiting the advance of sea water into fresh water aquifers. The effectiveness of such measures, as well as the design criteria on which they might be based, was essentially unknown when in 1951 the California State Water Resources Board initiated an extensive program designed to explore methods for the prevention and control of sea water intrusion. A part of this program was carried out by the use of scale models in the laboratory and is the subject of this report.

Purpose and Scope of Study

The investigation was intended to include studies involving:

1. An abstract of literature pertaining to sea water intrusion and its control.

2. Parameters of sea water intrusion, such as shape and rates of travel of interface between intruding saline waters and displaced fresh water.

3. Shape and height of pressure ridge, and required height of ridge to prevent sea water intrusion as related to such important variables as: depth, permeability, and thickness of aquifer; difference in specific gravity of liquids; and rate of injection.

4. Methods of construction and type of well best suited for injection purposes.

5. Hydraulics of injection and of injection wells as related to hydrologic variables.

6. Effect of relative depth of penetration of injection wells into aquifer.

7. Reduction in aquifer permeability through use of emulsions, silica gel, plastics, and other materials.

8. Raising the elevation of a ground water surface by rearrangement of pattern of pumping.

To carry out these purposes it was expected that theoretical considerations and relationships should be developed in conjunction with experimental work. A lapse time 16mm color motion picture showing the behavior of an intruded sea water wedge under various conditions of fresh water recharge and ground water pumping was also prepared during the course of the study.

Organization for Study

The studies herein reported were conducted by the Sanitary Engineering Research Laboratory in accord with the terms of Standard Service Agreement No. 3 SA-423 between the California State Water Resources Board and the Regents of the University of California, and based on a research proposal by Professor T. R. Simpson and Professor Harold B. Gotaas. The original contract, dated January 1, 1952, was amended on December 10, 1952 to extend

the original closing date from December 31, 1952 to August 1, 1953.

Professor T. R. Simpson and Professor Frederick L. Hotes served as faculty investigators on the project. The investigative work was carried out by Mr. James A. Harder, Project Engineer. During much of the study he was assisted by Mr. Leung-ku Lau. Professor D. K. Todd prepared an abstract of literature pertaining to sea water intrusion, which was published in July, 1953 as Technical Bulletin 10, of the Sanitary Engineering Research Laboratory. The report was prepared by Mr. Harder and Professor Hotes.

Acknowledgments

Grateful acknowledgement is made to the numerous agencies and individuals whose contributions helped make this investigation possible. Special thanks are extended to the engineering staffs of the office of State Engineer and the Los Angeles County Flood Control District, who gave freely of their time and counsel; to members of the faculty of the Division of Mechanical Engineering and the Division of Civil Engineering and Irrigation of the University of California, for advice and for the loan of special equipment; and to the staff of the Sanitary Engineering Research Laboratory for aid in conducting the research. The authors are also indebted to Professor K. S. Pister, Ahmed Sami El-Naggar, and M. Nour Eddine Rifai, and B. K. Karoly for investigative work; and to P. H. McGauhey for editorial assistance in preparing this report.

INTRODUCTION

Sea Water Intrusion -- Historical

About one hundred years ago Braithwaite (1) described increasing salinity of water pumped from wells in London and Liverpool and suggested that the source was sea water infiltrating as a result of the ground water table being lowered below sea level. His suggestion provoked considerable controversy among engineers of that time who were by no means in agreement that sea water intrusion might occur under such circumstances.

In 1889, however Ghyben (2) discovered a principle which enabled him to state with certainty that sea water intrusion would take place when certain relationships existed between fresh water in an aquifer and sea water at the outcrop of the aquifer. Successful application of this principle to the water supply problem along the North Sea was reported by Herzberg (3) in 1901, when numerous test borings revealed relationships between fresh and salt water pressures approximately as predicted theoretically by Ghyben.

The most urgent concern with sea water intrusion and with methods for its control developed perhaps in the low countries of Northern Europe. Many of the most significant contributions to an understanding of the subject have come from experience in that area. d'Andrimont (4) in 1902 described sea water intrusion conditions on the Belgian coast and showed the relationship of sea water intrusion into an unconfined aquifer and a confined aquifer underlying it. In a subsequent study (5) he reached the conclusion that there was a tremendous quantity of water available in the dune area, that most of it flowed through the dunes toward the sea rapidly enough to prevent sea water intrusion, that a large volume of this water might be used without causing sea water intrusion, and that the water found in the dunes was potable. Van Ertborn (6) (7) however, reported that the water in the dunes was not potable and disagreed with d'Andrimont on several points. In 1905 Pennink (8) described investigations of fresh and salt ground waters in the coastal dune area of Amsterdam which supported the findings of Herzberg and d'Andrimont. In the same year DuBois (9) found variations from the Ghyben and Herzberg observations which he attributed to the peculiar topographic and geologic conditions of the Netherlands. He noted, however, an increase in salinity due to pumping.

Since that time the now familiar Ghyben-Herzberg principle, discussed in some detail in a later section of this report, has been observed by many individuals to hold in specific instances of sea water intrusion in the low countries and elsewhere. A number of such instances are summarized in a separate report (10) which grew out of the investigation herein reported.

Reliable observations of sea water intrusion have been reported from Japan (11) (12) (13), the Bahama Islands (14), and Hawaii and various islands of the Pacific. These observations give some evidence of the shape of a fresh water - salt water interface and in general lead to the conclusion that the rate of withdrawal from ground water storage in coastal areas should not exceed the rate of ground water recharge from rainfall.

Intrusion of salt water in underground basins in the United States has been variously reported. Brown and Parker (15) described salt water penetration of 8000-9000 feet inland as a result of lowering the water table by a drainage ditch in southern Florida; Brown (16) reported intrusions of from 250 to 700 feet inland into wells in Connecticut; and Turner and Foster (17) reported sea water intrusion at depths below 1000 feet for distances up to 20 miles inland in the Galveston, Texas area in 1934. Barksdale (18) reported in 1940 that salt water had encroached into a pressure aquifer in New Jersey a distance of two miles at a rate of one mile in six years. Cederstrom (19) in Virginia recommended that coastal wells be located a mile or more inland from the coast and be of sufficient number so that no large yields of individual wells are required.

In California expressions of considerable alarm over intruding sea water began to appear in the literature in the middle 1940's. Simpson (20) noted that in 1945 approximately 6000 acres in the Salinas Basin were contaminated by sea water extending up to 1-3/4 miles inland at a rate of some 600 feet per year. Poland (21) summarized the situation in southern California and suggested controlling sea water by such methods as balancing long-term basin-wide draft and replenishment; maintaining a

fresh water head above sea level inland from the saline front by regulated draft or artificial replenishment; dewatering through wells near the saline front but coastward from it; and constructing impervious subsurface dikes. Zander and Gleason (22) estimated that overdrafts in three basins in the Los Angeles area totalled 45,000 acre-feet as early as 1947. Various other writers called attention to these and similar situations.

The most recent summaries of conditions in California were presented by Banks et al (23) in 1950, and by Banks and Bookman (24) in 1951. At that time one of the most spectacular examples of sea water intrusion in California was in the Manhattan Beach area of the West Coastal Plain where a saline front had advanced inland a distance of more than 2 miles under a landward gradient imposed by ground water levels 5 to 15 feet below sea level. The reports showed that by 1950 all previously reported intrusions of sea water in California had progressed inland, and recommended experimental studies concerned with control of sea water intrusion by mounds of injected fresh or reclaimed water, and by impervious membranes.

Reduction of Aquifer Permeability

The use of earth or other cheap materials to form relatively impermeable membranes for the purpose of preventing or reducing seepage of water through soils is a very ancient practice. One of the best known applications is the construction of puddle cores within earth dams. The effectiveness of such and similar engineering procedures could not fail to suggest their application to the closing off of leaks through dams and levees, and ultimately to the prevention of movement of sea water into an aquifer from which fresh water has been removed. The successful use of bentonite clay in well drilling muds, and of various bituminous and cement mixtures in general soil stabilization made such materials worthy of consideration for the construction of impermeable membranes underground.

During the past 25 years numerous investigators (10) have described the properties of bentonite and its application to certain engineering use. As early as 1925 Wherry (25) presented a concept which might explain the swelling of bentonite in the presence of water. Warren (26) observed, however, that saturated salt solutions are not adsorbed by bentonite, and that hence the material does not swell in such solutions as it does in fresh water. He was able to take advantage of this fact in drilling a well through a bentonite layer. Davis (27) described the effect of various liquids and physical and chemical factors on the swelling of bentonite, and Ambrose and Loomis (28) studied both the swelling and gelling properties of the material and outlined methods for controlling the swelling. These and numerous other reports established the unique properties of bentonite and indicated its possible usefulness in preventing seepage of water. Extensive experiments designed to test the applicability of soil-bentonite mixtures to the practical control of seepage have been carried out by the Corps of Engineers (29) (30) (31) over a period of some 15 years. Some of the results indicated that a 12.5 percent bentonite grout is stable for hydraulic gradients less than 4.0, but the material was not recommended for use in a cut-off wall in a slag formation for which it was tested in one series of experiments.

Experience with other types of sealing agents is more extensive. Cement grout injected under pressure has long been used in sealing foundations and other localized strata. In recent years considerable use has been made of a patented asphaltic emulsion (Shellperm) for controlling underground movement of water. The first large scale test of Shellperm in the United States was the construction of a vertical underground barrier to prevent leakage through a diversion dam. As described by Blakeley and Endersby (32), injected asphalt emulsion placed along a 350-foot length in depths varying from 5 to 30 feet effectively limited ground water losses. Other experiments in the use of Shellperm are described by Endersby (33) (34) (35). Hefley and Cardwell (36) described the use of liquid plastics as plugging agents for making aquifers impermeable to water in oil fields.

In the foregoing, and in most of the cases in which underground movement of water has been restricted by sealing agents, the practice has been to inject the sealing agent under pressure into closely spaced grouting holes. Serious doubts have been expressed, concerning the economic applicability of such a method to halting sea water intrusion along an extensive line across a deep aquifer. Some reports of placing cutoff walls in trenches, however, appear in the literature. The construction of 6.3 miles of sublevee cutoff trenches ranging in depth to 60 feet along the Columbia River is reported (37). Rodes (38) describes the placing of a puddled clay cutoff wall around an oil field near Long Beach, California. The puddle was placed in a 32-inch wide trench dug with a ladder type trenching machine followed by continuous backfill. Although the depth of cut capable with this machine was but 45 feet, there is sufficient agreement that much deeper cuts would be economical to justify serious studies of impervious membranes as devices for preventing sea water intrusion.

THE INVESTIGATION

I. STUDIES OF SEA WATER INTRUSION.

FUNDAMENTAL CONSIDERATIONS.

Occurrence of Sea Water Intrusion

Most of the water-bearing formations along the California coast are in direct contact with the floor of the ocean or an inland bay. Where this situation exists, sea water intrusion is a present or potential th[illegible] the associated ground water basin. Before these basins were exploited, a seaward hydraulic gradient existed and excess fresh water from inland areas escaped from surface springs near the beaches, or in the case of confined or pressure aquifers, from submarine springs off the coast. The recent rapid development of these basins has in some instances lowered the water table or piezometric surface to the extent that the fresh water pressure is less than the pressure exerted by sea water at pumping depths. Where this situation exists sea water may be expected to start moving inland under the influence of the reversed hydraulic gradient.

Height of the Piezometric Surface Required to Prevent Intrusion

To describe the minimum elevation of the fresh water table or piezometric surface required to prevent sea water intrusion in a quantitative way the difference in density or unit weight between fresh and salt waters must be taken into consideration. Most hydraulic engineers think of aquifer pressure in terms of feet of "head"; however, unless the usual definition is modified, the term "head" should be restricted to mean "fresh water head." A certain head of sea water or of any other fluid having a density different than fresh water is not equivalent to the same head of fresh water in producing fluid pressure, and at the interface between fresh and salt water it is the relative pressure (within a horizontal plane) which determines the direction of movement.

Theoretically, an intruded saline wedge can be held in a stationary position when the fresh water table is maintained at the proper elevation above mean sea level. Model studies have shown this to be true. Figure 1 shows an idealized section through a pressure aquifer subject to sea water intrusion. H represents the distance below sea level to the lowest pumping level which must be protected from damage. Under equilibrium conditions sea water is prevented from advancing further inland, and there is no energy gradient within the intruded sea water wedge to provide movement. This means that the fluid pressure within the wedge is hydrostatic and that the pressure at any point is the same as that existing at the corresponding ocean depth.

The pressure at a point on the interface, then, is equivalent to that produced by a column of sea water extending up to sea level.

In order to produce the same pressure on the fresh water side of the interface, a similar fresh water column, because of its lower density must extend above sea level. This distance (h in Figure 1) is equal to (S-1) H where S is the specific gravity of the sea water relative to the fresh water. This is the principle discovered by Ghyben (2) in 1889, and later applied to the water supply problem along the North Sea coast by Herzberg (3) as previously mentioned.

Length of the Intruded Saline Wedge under Equilibrium Conditions

Since it is necessary to hold the water table or piezometric surface above sea level to maintain the sea water wedge in a given position, it might be expected that this would result in a certain leakage of fresh water to the ocean through other strata. For if the fresh water pressure is sufficient to hold back sea water at one depth, according to the above principle it will be more than enough to hold it back at lesser depths. Thus, if the aquifer has a finite thickness, fresh water will push seaward in the upper portion under the same head that is required merely to hold the sea water stationary in the lower portion. Since an estimate of the fresh water leakage which may occur under these conditions may be of value, an expression has been derived relating leakage to other aquifer characteristics:

$$q = \frac{1}{2}\ (S-1)\ \frac{M}{L}\ T \qquad \text{Eq. (1)}$$

in which (see Fig. 1):

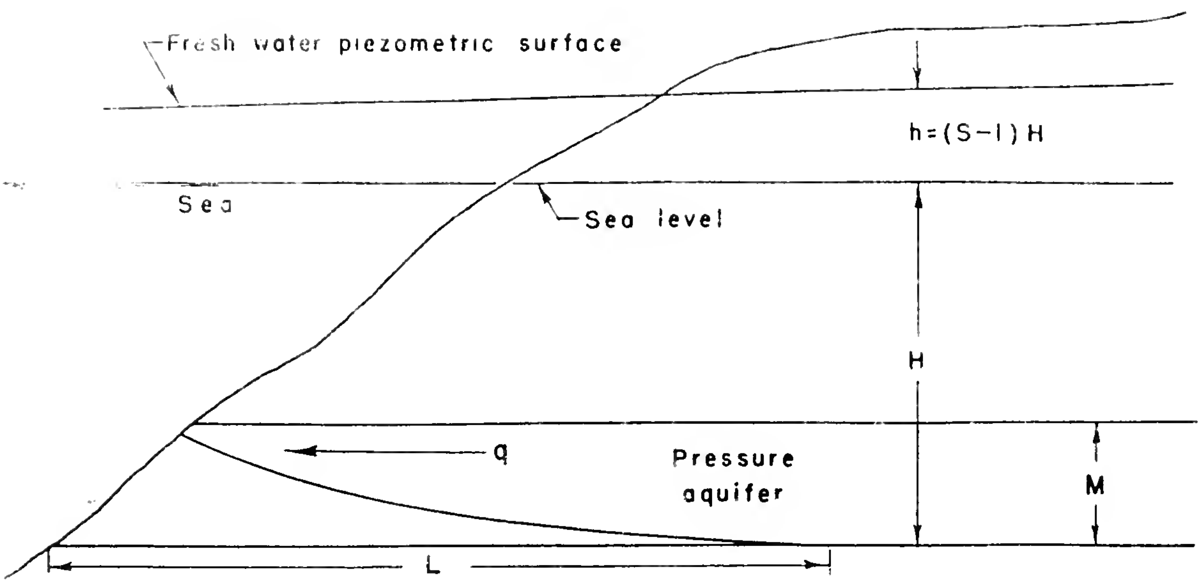


Figure 1. Schematic Drawing of a Section Through a Pressure Aquifer.

q = seaward fresh water flow per foot of ocean front

S = specific gravity of sea water

M = thickness of aquifer, down to the lowest depth which must be protected

T = aquifer transmissibility for a 100 percent hydraulic gradient

L = length of sea water wedge, from ocean outlet to the toe

The derivation of this equation is given in Appendix I. It is based on certain idealized conditions, assumptions, and approximations. Nevertheless, experiments (see Fig. 9) verify its basic validity, and when applied with good engineering judgment it can be quite useful in the study and solution of sea water intrusion problems.

Hydraulics of Injection Wells

One proposed method for controlling sea water intrusion into pressure aquifers is the operation of a line of injection wells parallel to the coast. There is no doubt but that a pressure zone can be built up in this way; however, the quantity of water which must be injected or the well spacing required may place restrictions on its practicality. In order to help interpret the results of model studies of such a system, certain basic considerations will be developed from the theory of potential fields.*

The steady state flow of water in a uniform confined aquifer is governed by the Laplace equation with the potential represented by the height of the piezometric surface. Of course, no natural aquifer is uniform either in thickness or permeability, but for the purpose of developing general principles such an aquifer is often assumed. After the general principles are understood, the effect of non-uniformities can be more intelligently evaluated.

In the following analysis a uniform confined aquifer into which a long line of equally spaced injection wells has been introduced is assumed. Further, to simplify the mathematical treatment, it is assumed that at each end the line of wells terminates at a lateral boundary, such as might be the case if the wells were drilled across the mouth of an alluvial valley. Under such assumptions the steady state distribution of pressure or potential within the aquifer is governed by the Laplace equation in two dimensions and can be determined when either the height or the slope of the piezometric surface is given within a suitable boundary zone.

If the problem is further simplified by assuming that the injection rate is the same for all wells, the potential distribution can be readily obtained.

Figure 2 illustrates the equipotential contours associated with an equal flow in each direction from the injection line. For an unequal flow, the equipotential contours are distorted from the

* See Appendix II for development of theory.

equal-flow pattern. Figure 3 illustrates such a condition when three-fourths of the flow is in the positive Y direction.

Within a short distance, approximately half the well spacing beyond the centerline of the wells in either direction, the equipotential lines become virtually parallel. This indicates that in the outer region the flow lines, which are perpendicular to the equipotential lines, take a direction very nearly the same as they would were the injected water introduced from a line source coincident with the centerline of the wells. If Q represents the injection rate per well, then the uniform flow in each direction is $Q/2a$, where a is the well spacing.

A non-symmetrical flow pattern may be developed by superimposing a hypothetical uniform flow normal to the centerline of the wells on a

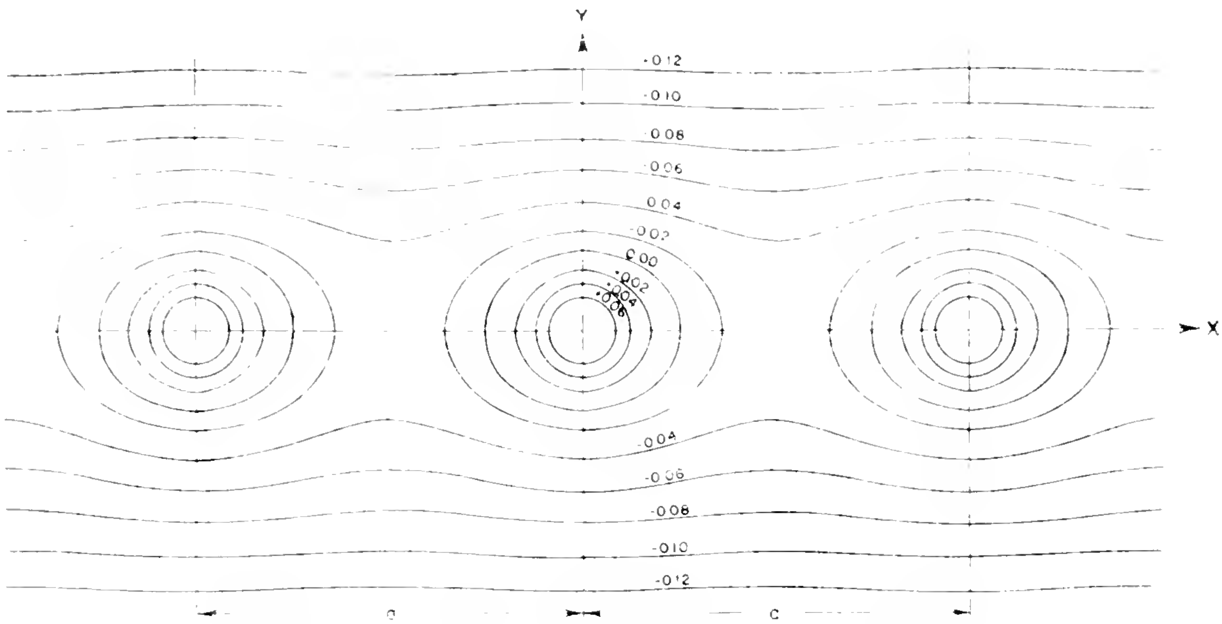


Figure 2. Equipotential Lines Associated With a Line of Injection Wells. Equal Flow in Each Direction.

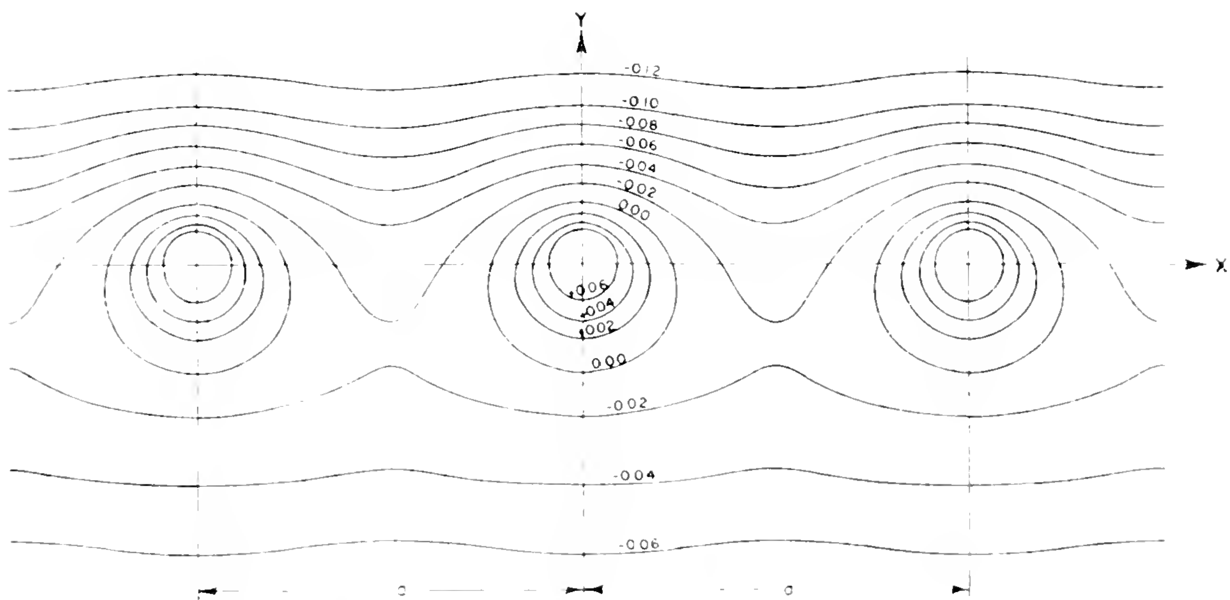


Figure 3. Equipotential Lines Associated With a Line of Injection Wells. Three Fourths of the Flow in the Positive Y Direction.

symmetrical flow from the wells. Let q be the uniform transverse flow per unit width, q_i be the actual flow inland (in the outer region, one-half well spacing beyond the wells), and q_s be the actual flow per unit width towards the sea. Assume that positive values of q represent a flow in the inland direction. Thus

$$q_s = \frac{Q}{2a} - q \qquad \text{Eq. 2a}$$

$$q_i = \frac{Q}{2a} + q \qquad \text{Eq. 2b}$$

Solving for Q and q:

$$Q = a\,(q_i + q_s) \qquad \text{Eq. 3a}$$

$$q = \frac{1}{2}(q_i - q_s) \qquad \text{Eq. 3b}$$

For known values of q_i and q_s, Q and q can be determined from equations 3a and 3b and equipotential contours similar to those of Figures 2 and 3 can be constructed according to the method described in Appendix II.

Equation 3a shows that the required injection rate per well must equal the sum of the seaward and inland flows, per foot of front, multiplied by the well spacing. This is an intuitively obvious consequence of the principle of continuity.

Sea water intrusion is ordinarily associated with an overdraft condition. As fresh water is withdrawn from inland areas, sea water encroaches from the coast and actually helps maintain pressure within the aquifer. The flow is continuous and the interface between the fresh and salt water gradually moves inland. Since it is unlikely that raising the piezometric surface near the coast will reduce the inland flow, any existing inland flow due to overdraft conditions will continue to take place after an injection process has been started, unless inland pumping is reduced.

MODEL STUDIES

The Model Aquifer

The model used in experimental studies is shown in Figures 4 and 5. It was constructed of Lucite (Plexiglass) to allow a visual observation of the position and movement of the sea water-fresh water wedge. The end chambers, separated from the central section by fine bronze screens, were originally designed to bolt to any of several center pieces of various widths to a maximum of twelve inches. All sections were to have the same 6-inch height, and four-foot length, variation in width making possible the study of an injection well system with different ratios of well spacing to aquifer thickness. The first

Figure 4. General View of the Model.

center section constructed was three inches wide. (see Fig. 5) Subsequent experience with this model indicated that no information of any importance could be gained from increasing the width. The central section was filled with sand and equipped with piezometer tubes for establishing the piezometric surface under any condition of flow. One end chamber was connected to a constant head sea water reservoir through a flow regulator by means of rubber tubing. Similar devices fed fresh water into the other end chamber.

Although the model was designed for a generalized study of sea water intrusion into different aquifers, scale factors were arbitrarily assigned to various model quantities to provide a feeling for the prototype quantities they might represent. Thus the horizontal scale factor was assumed to be 1:2000 and the vertical scale factor 1:200. On this basis the model would represent a section of confined aquifer 100 feet thick and 8000 feet long. Actual data are recorded in terms of dimensionless ratios, and so the information obtained is more general than these values would indicate. They do, however, give a better concept of what the model data might mean under a certain set of prototype conditions. For the above horizontal scale factor, the model width of three inches would correspond to a five hundred foot wide section of aquifer, the width being in a direction parallel to the coast. If a model injection well were located in the center of this strip, it would represent a section of an aquifer into which a long line of equally spaced injection wells had been drilled at 500 ft. intervals.

Actually, the model injection wells were located along only one side of the aquifer. This made it possible to observe the interface movement in each of two different vertical planes corresponding to planes of symmetry in the

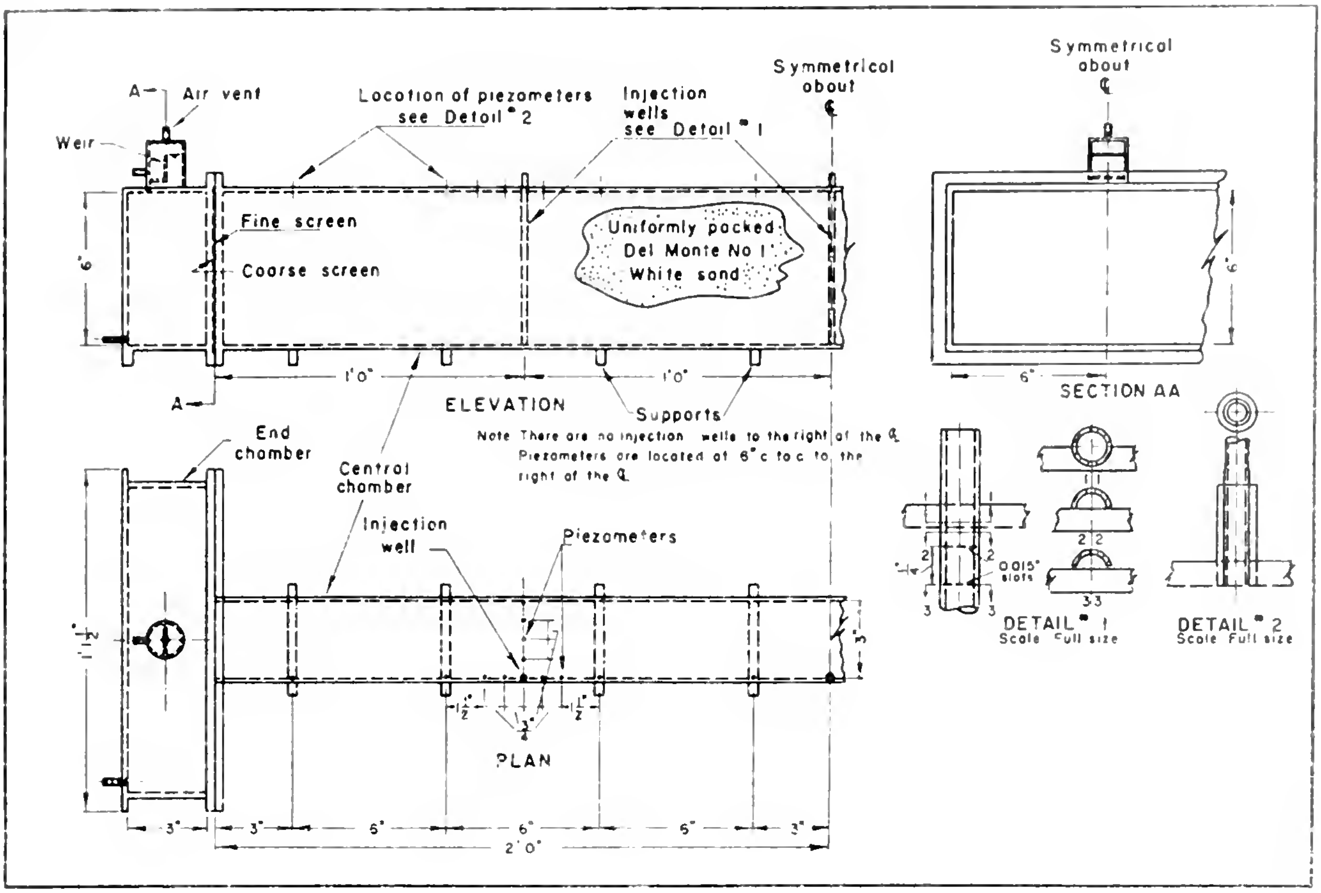


Figure 5 Lucite Model Aquifer.

prototype. One of these planes passes directly through an injection well while the other passes midway between two injection wells. No flow crosses these planes under a uniform injection process, so that they can be represented by solid walls in the model. On this basis the model represents a half section of an aquifer into which injection wells have been drilled at 1000 ft. intervals.

The model injection wells, one-fourth inch in diameter, correspond to a prototype well approximately forty feet in diameter on the basis of the above scale factors. This enormity is not as serious as it might at first appear, for the location of the piezometric surface contours depends on the water flowing from the well and on boundary conditions, such as the quantity of water flowing towards the ocean or towards the inland area. The water level observed in a model well, then, corresponds to the piezometric surface which would be observed approximately twenty feet away from its prototype. Within this twenty-foot radius zone, the flow pattern is for all practical purposes radial, and the height of the piezometric surface in the prototype may be computed analytically from the injection rate and the aquifer transmissibility in the region. The water level within the prototype injection well will depend on its effective diameter, and can be computed using Equation II-7 wherein x may be set to equal the effective radius and y set to zero. Coefficients for this equation are determined from boundary conditions as shown in Appendix II. This analysis does not take into consideration additional resistance to flow which may result from localized clogging near the injection well casing.

Model Aquifer Sand

The model aquifer was packed with a commercially available washed quartz sand, Del Monte 30 mesh. This sand has an effective size (10%) of 0.22 mm, a 50% size of 0.28 mm, and a uniformity coefficient (60/10) of 1.3. Its coefficient of permeability averaged 0.032 cm/sec, as determined on sand packed within the model. The low uniformity coefficient made is possible to pack the model without developing noticeable segregation. The packing procedure itself was designed to minimize any tendency toward the formation of horizontal layers. Since the center section was essentially a hollow rectangular tube, it could be tilted up on one end with the lower end closed off by a screen and one of the

end sections. In this position it was partially filled with water, the water level being maintained slightly above the level of the sand as it was deposited through the open end and vibrated into place. In this way air-entrainment was avoided and whatever stratification that tended to occur was in a vertical plane when the model was returned to its normal position.

Since it would be impossible to duplicate the infinite variety of non-uniformities found in a natural aquifer, only the simplest case, that of a completely uniform aquifer, was studied on a model scale. Some basic principles found for this case can be applied directly to a non-uniform aquifer; in any instance, however, the knowledge of flow conditions within a uniform aquifer should form a valuable basis for estimating what changes in the flow pattern may be expected due to the presence of non-uniformities. The effect of such non-uniformities on the quantity of seaward flow required to stabilize the sea water wedge has been discussed in a previous section

Thus no attempt was made to correlate the sand size and grading within the model to values which might be found in a prototype aquifer; the permeability itself, which includes these factors implicitly, is the only property of the porous medium which determines the flow rate.

Model Representation of Sea Water

The salt water used in the model was adjusted to a specific gravity of 1.100; this made the scale factor 3.85 based on a normal sea water specific gravity of 1.026. Since the scale factor for flow rates is directly proportional to the scale factor for relative specific gravity **(Eq. 1)**, this procedure increased the model flow rates and generally speeded up an otherwise very slow action. An even greater difference in the specific gravity would have been desirable on this basis, but higher concentrations of salt would have increased the viscosity of the solution to a serious extent.

Both calcium chloride and sodium chloride solutions have been used to simulate sea water. The solutions of these salts at a specific gravity of 1.10 have a viscosity, respectively, 38 percent and 23 percent greater than that of pure water. Normal sea water at a specific gravity of 1.026 has a viscosity approximately 7 percent greater than that of pure water. This discrepancy does not affect measurements of the equilibrium position of the interface when there is no movement of the sea water wedge; also it cannot affect the general movement of the interface under a simulated inland overdraft, for this depends only on the flow rate and the effective cross sectional area.

Shifts in the shape of the interface must be affected to some extent. However, the uncertain effects that sea water cations can have on the permeability of a natural aquifer may well be more variable than this introduced error when the performance of the model is compared with such an aquifer. Sand used within the model was carefully washed free of clay and there is no reason to expect that its permeability would depend on the ionic concentration.

The salt water was distinguished from the fresh by introducing dyes of different color into the two phases. Since a convenient concentration of most dyes does not show up very well in the model because of the thin layer of solution visible between the transparent side and the first layer of sand grains, it was found expedient to use flourescent dyes. An ultra-violet light source was used to produce a strong and positive response from the thin layer of solution next to the wall. Rhodamine B in the saline water and Fluorosol in the fresh water gave good contrasting colors of red sea water and blue fresh water.

Model Scale Factors

The essential requirement demanded of a set of model scale factors is that they relate the operation of a physical law in the prototype to its operation in a model of reduced dimensions. This means that equations expressing physical laws must apply equally to prototype and model quantities.

The basic flow equation, $q = \frac{1}{2}(S-1)\frac{M}{L}T$

(Eq. 1), is the basis for the calculation of other model ratios.

For example, the ratio of the prototype seaward flow to the model seaward flow is determined as follows.

Let:

K = ratio of $\frac{\text{prototype property}}{\text{model property}}$

The subscript	z	refer to	horizontal dimensions	
"	y	" "	vertical	"
"	$S-1$	" "	(Sp.Gr.-1)	"
"	p	" "	permeability	"
"	t	" "	time	"
"	q	" "	quantity	"

Equation 1 is first rewritten by replacing transmissibility with the product of permeability and aquifer thickness, which define transmissibility. That is:

$$q = \frac{1}{2}(S-1)\frac{PM^2}{L} \qquad \text{Eq. 1a}$$

$$\text{Then } Kq = \frac{K_p \times K_y^2 \times K_{S-1}}{K_z}$$

Now, for the scale ratios:

$K_z = 2000$ (horizontal acale)

$K_y = 200$ (vertical scale) Eq. 4

$$K_{S-1} = \frac{1.026-1}{1.1-1} = \frac{0.026}{0.1} = 0.26.$$

For $K_p = 2$ (assumed):

$$K_q = \frac{2(200)^2(0.26)}{2000} = 10.4$$

This means that the discharge per foot of aquifer width would be 10.4 times as great as the discharge through a foot of model width.

A typical time scale would be computed as follows:

$$K_q = \frac{K_z K_y}{K_t} = \frac{K_p \times K_y^2 \times K_{S-1}}{K_z} \qquad \text{Eq. 5}$$

Solving for K_t:

$$K_t = \frac{K_z^2}{K_p K_y K_{S-1}} = \frac{2000^2}{2(200)(0.26)} = 38{,}500 \qquad \text{Eq. 6}$$

This means that prototype action taking place over a period of 38,500 minutes (26.7 days) would take place in one minute in the model. It should be noted that for certain applications this time scale may be only approximate. Because the basic equation was derived only for equilibrium conditions, it cannot necessarily be applied with complete accuracy to the time required for the wedge to move from one position to another. It is believed that the error involved is small in view of the assumptions used.

In the foregoing analysis the horizontal and vertical scale factors have been purposely separated. Since the thickness of a typical pressure aquifer is small relative to its lateral extent, it is almost essential to exaggerate the vertical scale of the model of such an aquifer to observe the movement within it. For example, if a 8000-ft. long section of a 100-ft. aquifer were to be modeled within a four-foot length at the same horizontal and vertical scale ratios, the model would be only five-eights of an inch high. Actually the model was made four feet long and six inches high in order to photograph more easily the interface. This would introduce a tenfold vertical scale distortion on the basis of the foregoing illustration.

The analysis is based on the assumption that vertical velocity components within the porous medium are negligible. Even with a tenfold vertical scale exaggeration these components as observed in the model are small compared with the horizontal components, except in some instances near an injection well. In each instance where the vertical component of flow might be expected to exert some favorable influence, such as the case in which water injected through perforations near the bottom of an aquifer might be expected to produce a higher relative pressure there, the scale exaggeration in the model should tend to accentuate the desirable effect. Evidence that such desirable phenomena are not present in a distorted model should then be an even stronger indication that they would be absent in the prototype.

Recording Equipment

It seemed desirable that a motion picture be made to record the behavior of the saline wedge in the model aquifer under various test conditions. The necessity for using fluorescent dye under ultra-violet illumination in order to observe visually the movement of the wedge imposed problems of photography. In addition, the movement of the wedge was quite slow, making it imperative that pictures be taken by a lapse-time technique. By using Rhodamine B in the saline water and Fluorosol in the fresh water it was possible to obtain a good record on kodachrome film under ultra-violet illumination.

The lapse-time equipment consisted of a 16-mm. Kodak Cine Special operated by a special motor and timer mechanism (Fig. 6). A 6 rpm electric motor took the place of the manually wound drive mechanism originally furnished with the camera. Various exposure times for a

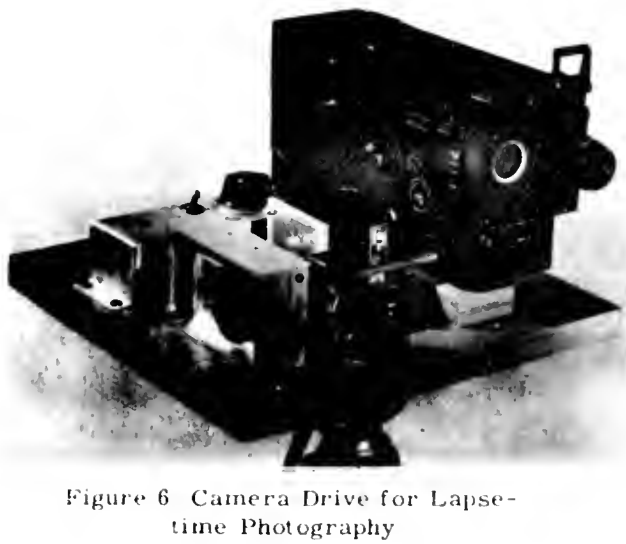


Figure 6 Camera Drive for Lapse-time Photography

single frame were achieved by means of different combinations of gears in the power transmission between the motor and camera. The lapse-time between single frame exposures was controlled by a timer, which could be shut off to permit the camera to run continuously. A relay associated with the camera motor automatically turned the room lights off during camera operation and on during the interval between frames. Another accessory was a clock-operated switch which would shut off the electricity to the entire assembly at any preset time; this could be used to extend the camera operation beyond normal working hours.

The resulting motion picture, edited and subtitled, shows graphically the behavior of an intruded saline wedge under a variety of conditions.

Flow Measurements

In spite of the vertical scale exaggeration, the direct determination of relative piezometric surface elevation in a model of the size used presents a difficult problem; the datum of each measuring tube must be continuously rechecked, since a very small error in determining the elevation of two points can produce a large error in determining a small slope between them. In practice it was found more satisfactory to find any needed hydraulic gradients from the measured flow through the model aquifer and from its transmissibility. The transmissibility itself can be measured at relatively large hydraulic gradients and rates of flow, and in practice was rechecked frequently.

Flow measurements were made with the apparatus illustrated in Figures 7 and 8. The fluid to be metered was led from a constant head tank through the tube entering from the right to the plunger type flow regulator. The glass rod fits the barrel of the regulator closely, and by varying the length of the annular space thus formed very small rates can be measured with adequate accuracy. The manometer in the line leading to the model (left) measures the back pressure on the regulator; in practice the outlet of the sampling nozzle is maintained at the level of the manometer meniscus. Thus when the two-way valve is turned to divert the flow for measurement the same pressure difference is maintained across the regulator. In this way the flow rate could be determined volumetrically in ten or twenty seconds, using a 10 ml graduated cylinder and stopwatch; quickly enough to avoid any appreciable effect on interfacial movement within the model.

Diffusion at the Sea Water -- Fresh Water Interface

In addition to allowing an easier observation of the interface, the vertical scale distortion employed made it possible to relate diffusion phenomena in the model and prototype. Of course there is no way to reduce the constants of molecular diffusion by any scale factor, since they reside in the nature of the diffusing material. The relative proportions of the diffusion zone between the fresh and salt water phases can be maintained, however, through an appropriate vertical scale distortion.

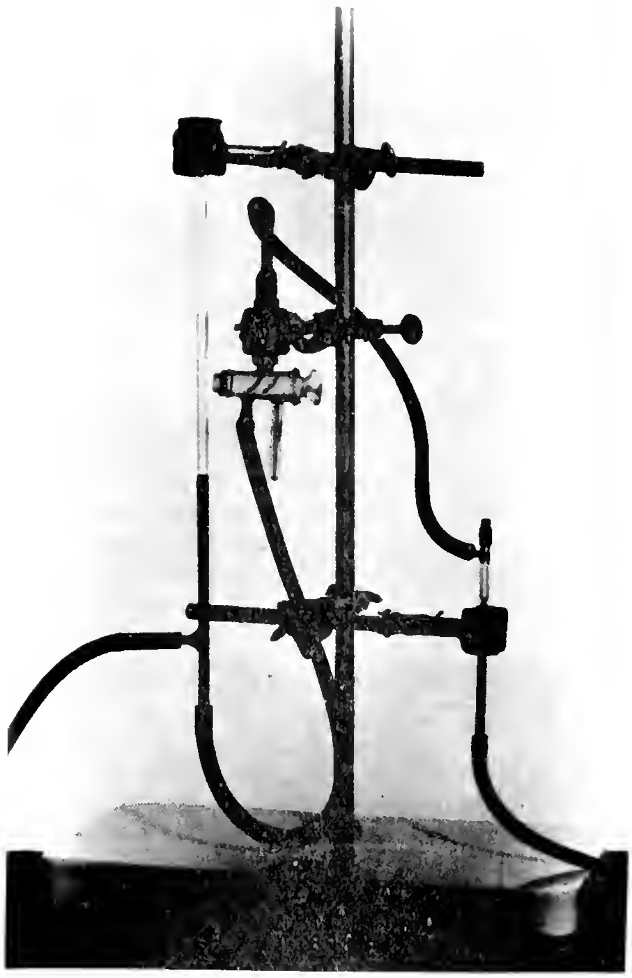
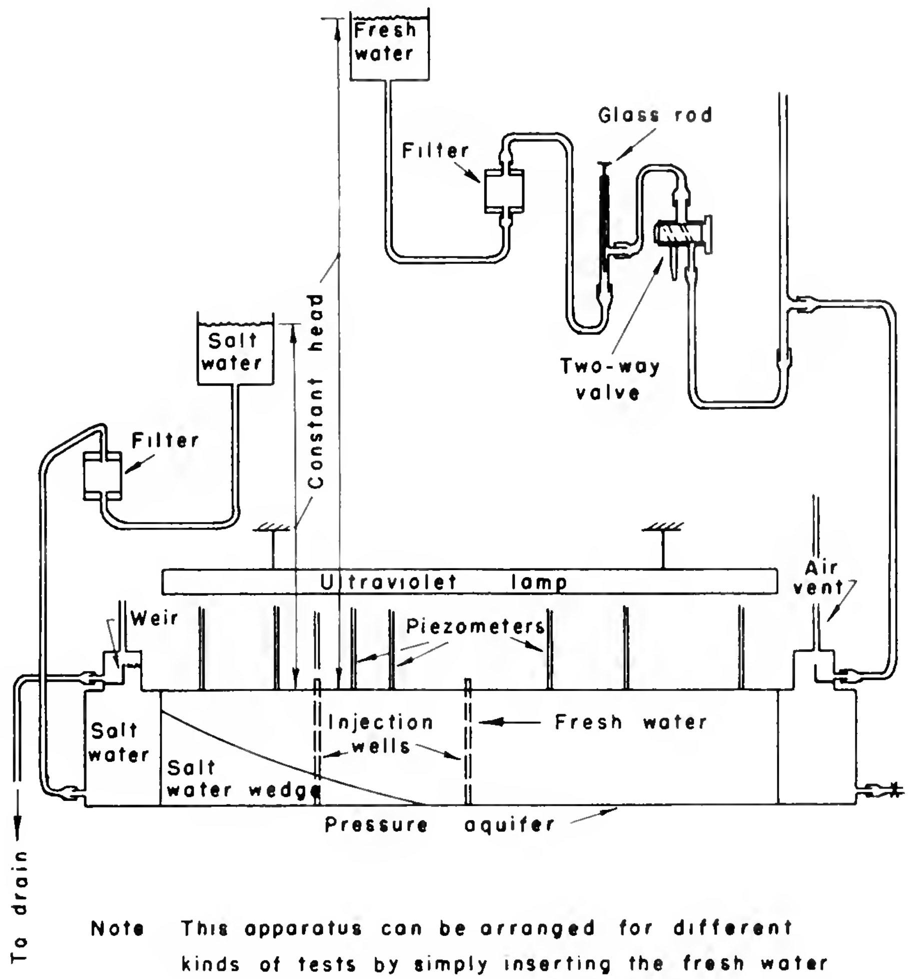


Figure 7. Flow Regulator

Under equilibrium conditions, when the sea water wedge is stationary, a constant flow of fresh water seaward over the wedge tends to sweep any diluted sea water resulting from upward diffusion at the interface back out to the ocean outlet. This flow is essentially horizontal while the diffusion is essentially vertical. If the vertical scale is exaggerated the diffusion path is stretched out at the same time that other factors tend to increase the horizontal flow. These two effects together can serve to equalize the relative thickness of the diffusion zone between model and prototype.

SCHEMATIC SKETCH
of
APPARATUS



Note This apparatus can be arranged for different kinds of tests by simply inserting the fresh water and/or the salt water and/or the effluent at appropriate places on the model The above set-up shows fresh water being recharged from an inland area.

Figure 8. Schematic Sketch of Apparatus.

The well known differential equation governing diffusion in one dimension is

$$\frac{\partial c}{\partial t} = \lambda \frac{\partial^2 c}{\partial y^2} \qquad \text{Eq. 7}$$

in which c is the concentration, t is the time, y is the direction of diffusion, and λ is a constant for a given solvent and solute. This equation defines the relationship among the scale factors which must obtain for similitude of diffusion between model and prototype. Thus

$$\frac{K_c}{K_t} = K_\lambda \frac{K_c}{K_y^2} \qquad \text{Eq. 8a}$$

or

$$K_t = \frac{K_y^2}{K_\lambda} \qquad \text{Eq. 8b}$$

A relationship between the time scale factor K_t and the horizontal length scale factor K_z is already available from Equation 6. This may be rewritten in terms of transmissibility as follows:

$$K_t = \frac{K_z^2}{K_T K_{S-1}} \qquad \text{Eq. 6a}$$

where the subscript T indicates transmissibility.

Equating the time scale factors from Equations 6a and 8b

$$\frac{K_z^2}{K_T K_{S-1}} = \frac{K_y^2}{K_\lambda}$$

and solving for the required vertical scale distortion $\left(\frac{K_y}{K_z}\right)$ gives

$$\frac{K_y}{K_z} = \left[\frac{K_\lambda}{K_T K_{S-1}} \right]^{1/2} \qquad \text{Eq. 9}$$

The model transmissibility averaged 0.49 ml per cm per sec, which is equal to 340 gal per ft per day for a 100 percent hydraulic gradient. Assuming a prototype transmissibility of 150,000 gal per ft per day the volume of K_T is 442. The relative specific gravity scale factor, K_{S-1} was 0.26 based on a prototype salt water specific gravity of 1.026 (normal sea water). Introducing these values into Equation 9 we obtain for the required vertical scale distortion:

$$\frac{K_y}{K_z} = \left[\frac{1}{442(0.26)} \right]^{1/2} (K_\lambda)^{1/2}$$

$$\frac{K_y}{K_z} = [\text{illegible}]\, K_\lambda^{1/2} = \frac{K_\lambda^{1/2}}{10.7}$$

Since the diffusivity constant, λ, is of the same order of magnitude for most salt solutions it may be seen that a tenfold vertical scale distortion should give a good indication of the relative importance of the diffusion zone. The interface between the sea water wedge and overlying fresh water remained quite sharp in the model and such should also be the case for the prototype. From the results of field investigation in the Netherlands (39) it has also been concluded that the zone of diffusion is of negligible thickness. It should be noted in this connection that vertical diffusion within a well casing may be expected to be much greater than in the surrounding aquifer. Of course, when a well is pumping from different strata, some of which are contaminated with sea water, there is a natural mixing process. This, together with the fact that the intrusion generally invades the lower portions of an aquifer first, explains the apparently gradual increase in groundwater salinity under conditions of sea water intrusion.

Mixing at the Sea Water — Fresh Water Interface

A type of mixing distinct from diffusion occurs at the interface between sea water and fresh water whenever the interface shifts. This is due to the fact that a part of one phase is not displaced as readily as the rest. On a microscopic scale, a film of the displaced phase may be left behind on the sand grains while most of the phase is swept forward. Of more importance in an actual aquifer, however, is the fact that the interface may move more rapidly in some strata than in others under the same hydraulic gradient. It may be expected that under the influence of a landward hydraulic gradient the more permeable (and more useful) portions of the aquifer will be damaged first. Conversely, intruded sea water should be more easily displaced from the more permeable portions when the overdraft condition is corrected.

From observations of interfacial movement within a uniformly packed model aquifer it might be concluded that mixing due to interfacial movement is not more extensive than diffusion. However, no similar analysis has been made to show that this phenomenon can be quantitatively studied on a model scale. Some evidence that the mixing is not extensive has been found from experience with prototype injection wells (40).

EXPERIMENTAL RESULTS

Equilibrium Position of the Sea Water Wedge

In a previous section a theoretical equation was presented relating the seaward flow of

esh water (per foot of front), q, to the aquifer ransmissibility, T, and thickness, M; the specific gravity of the sea water, S; and the length f the intruded sea water wedge, L. Experimental values of q/T are plotted against $\frac{1}{2}(S-1)\frac{M}{L}$ in Figure 9. The graph of the theoretical relationship Equation 1 is also plotted thereon and appears as a straight line. Most of the experimental points lie slightly below and to the right of this line. One explanation is that in the

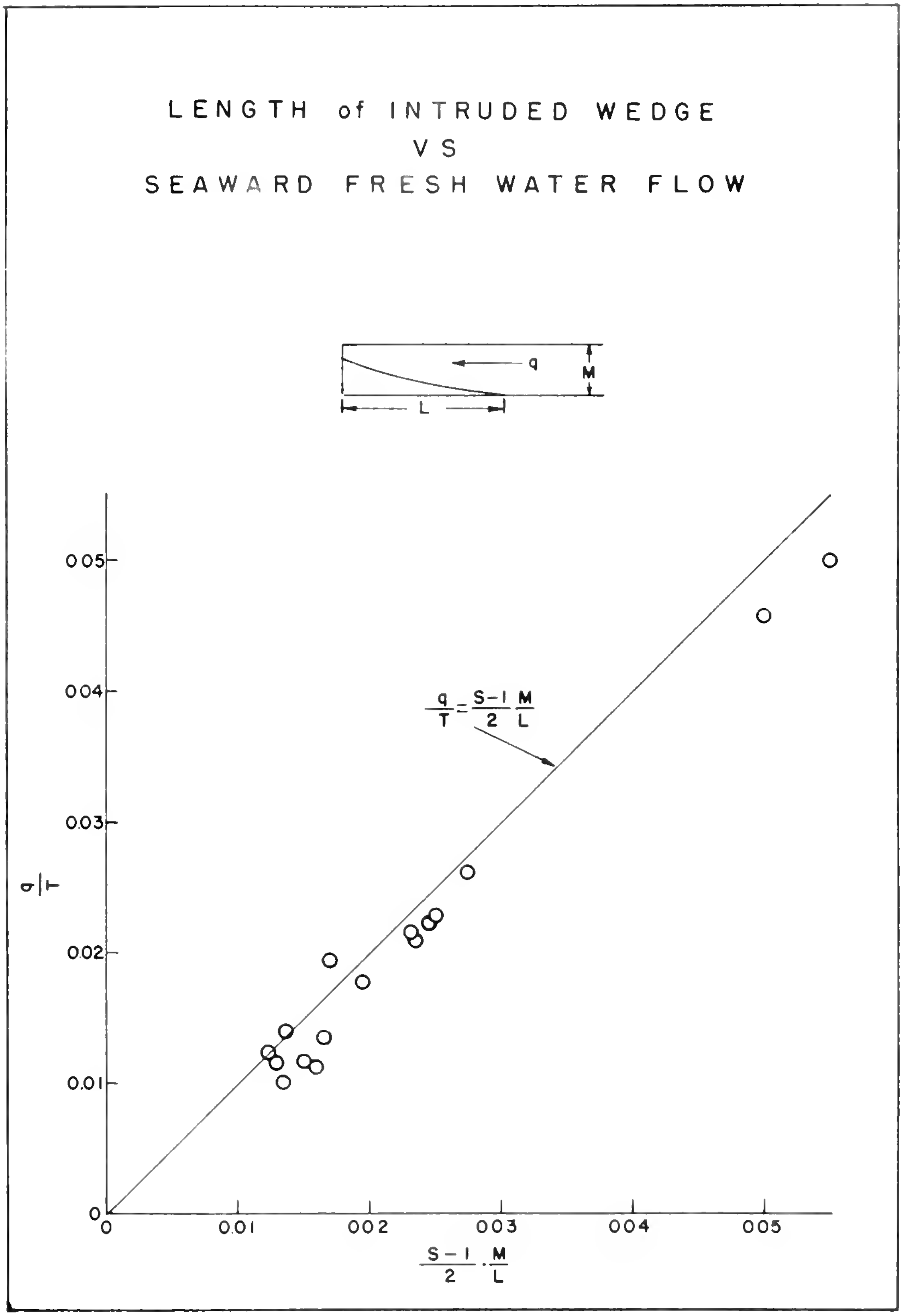
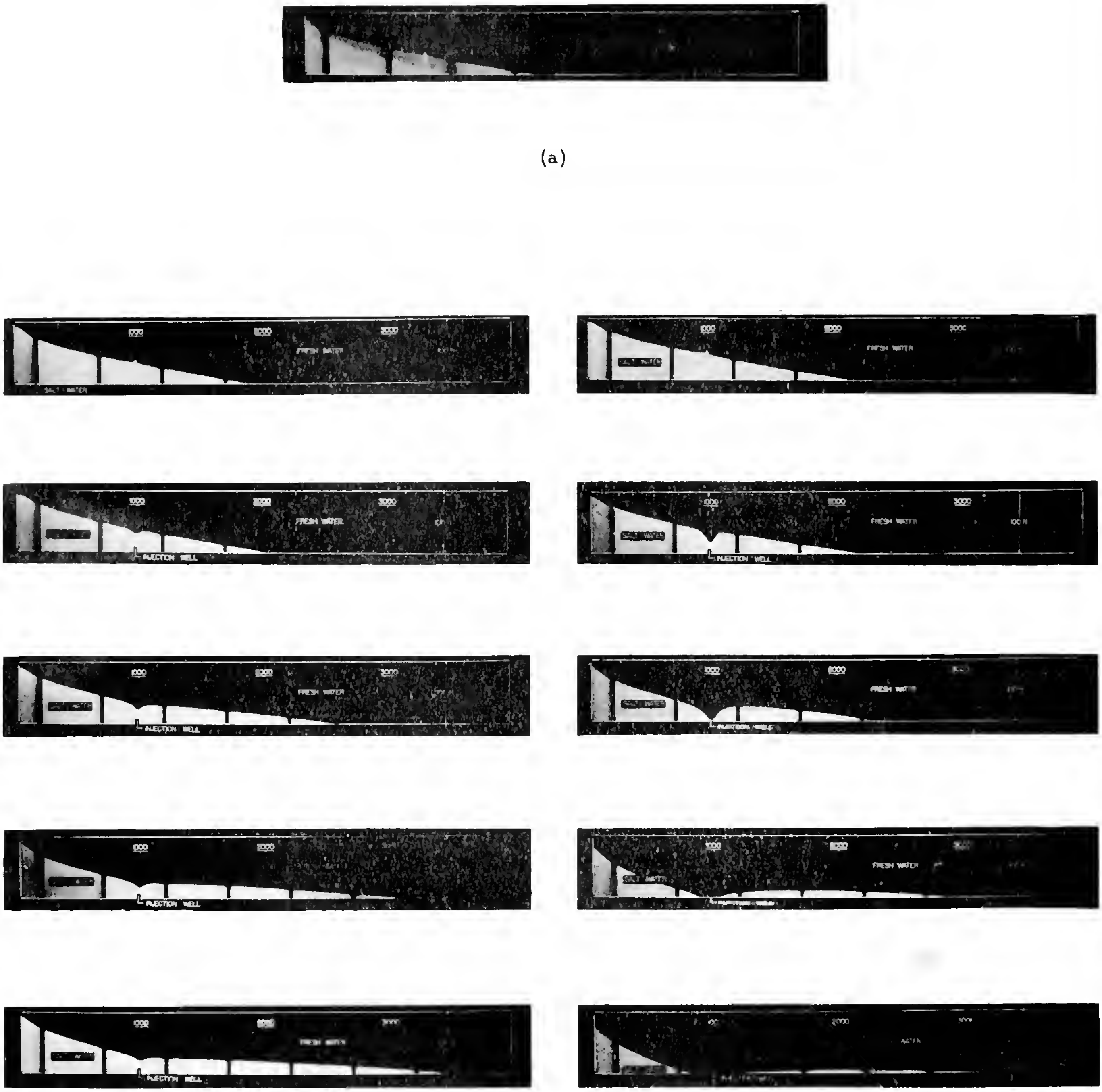


Figure 9. Length of Intruded Wedge versus Seaward Fresh Water Flow.



(a)

(b)

(c)

Figure 10. Behavior of Sea Water Wedge at Various Injection Rates.

experimental procedure the equilibrium position was approached from positions of the wedge representing larger values of the ratio M/L, so that unless a very long time were allowed the final length of the wedge might still be somewhat short of true equilibrium. Errors associated with measuring small flow rates and with maintaining a precise value of salt water specific gravity may account for the discrepancy between the plotted points and the curve of the equation. Also, despite precautions, the sand could not be maintained as tightly packed against the top plate of the model as against the sides and bottom. Any extra leakage along this surface would tend to upset the assumptions under which Equation 1 was developed and to increase the value of the coefficient to above 1/2.

A typical photograph of the sea water wedge at equilibrium corresponding to a particular flow rate is shown in Figure 10a.

Effects of Non-uniform Permeability on Seaward Leakage

In estimating the seaward leakage expected in a natural aquifer for a given location of the sea water wedge, Equation 1 must be modified for non-uniformities in the aquifer permeability. The factor 1/2 which appears in the equation takes into consideration the fact that as the flow approaches the ocean outlet a smaller cross sectional area is available due to the presence of the intruded wedge, which reduces the flow that would otherwise take place. It can be seen that if the transmissibility of the aquifer is greatest near the upper boundary or in other locations above the wedge, the presence of the wedge would have a smaller effect on the flow rate. Under this condition the factor would approach, but could not exceed, unity. On the other hand, if the transmissibility is greatest in the lower portion of the aquifer, the value of the coefficient may be less than one-half. A reasonable estimate of the value of the coefficient can be made from a knowledge of the relative permeabilities of strata within the aquifer. Only relative values are required, which can be obtained from core samples. The transmissibility itself is most accurately determined from pumping tests.

Another type of non-uniformity obtains if the permeability in the seaward extension of the aquifer differs from that in the coastal regions. In this case the transmissibility to be introduced into Equation 1 must be a composite value. A determination of the appropriate transmissibility may be made in some coastal regions from the slope of the piezometric surface between wells where the transmissibility can be measured. The ground water flow rate determined in this way can be related to the difference between the piezometric surface at the coastline and mean sea level to obtain the composite transmissibility of the off-shore extension of the aquifer. If the aquifer is partly filled with sea water, precautions must be taken to correct the piezometric surface determination for the density of the fluid in the measuring wells. A further complication arises in this latter case, the cations of normal sea water may either decrease or increase the permeability of an aquifer through the effect on whatever clay may be present.

Location of Injection Wells

The source of the seaward fresh water flow required to stabilize the sea water wedge is immaterial as long as it is located inland from the wedge toe. If a line of injection wells is located at least half a well spacing inland from the toe, seaward flowing fresh water from the wells will have merged to nearly a uniform flow as it reaches the toe and for all practical purposes will act as if it had come from a distant inland area. In this latter case the opportunity would exist for allowing the equilibrium position of the wedge to shift some distance inland where it could be maintained with a smaller fresh water flow rate. If the required flow is to be obtained from injection wells, however, the flow rate must be sufficient to hold the sea water wedge seaward from the line of wells, which places a limitation of the maximum length of intruded wedge which can be allowed and thus fixes the minimum flow which must be maintained. If the seaward flow from the injection wells does not meet this minimum requirement, sea water can pass between the wells and will be subject to whatever hydraulic gradients exist inland from the injection wells.

In the series of photographs of the model shown in Figure 10b there is no flow to or from the inland area represented by the chamber on the right hand end. Fresh water is injected through a well 1000 feet from the ocean outlet at a rate which would be sufficient to hold the sea water wedge at 2000 feet, were it flowing from the inland area. Since the flow is not sufficient to hold the wedge seaward from the well, however, sea water moves between the wells and enters the inland region as a density current. The injected fresh water, as well as the water displaced by the sea water density current, finds its way to the ocean outlet. Here we have an example of fresh water flowing in a direction opposite to that of the underlying salt water. The interface is shown in these photographs as in a plane twenty feet from a prototype injection well.

Figure 10c differs from Figure 10b in that the injection rate of fresh water is three times that in Figure 10b. This rate is sufficient to hold the wedge seaward from the well, or at 667 feet from the ocean outlet, so that the part of the sea water wedge inland from the well is completely cut off Under the condition simulated no fresh water flow takes place inland from the wells, but the isolated segment of the sea water wedge continues to flatten out under the influence of its greater density.

Rate of Interface Movement

A graph of the rate of interface movement at the wedge toe plotted against distance from the salt water chamber is shown in Figure 11. At the beginning of the experiment from which these data were taken the wedge toe had been stabilized at a position twelve inches from the salt water chamber. Then an overdraft period was simulated during which water was withdrawn from the fresh water chamber at a rate of q/T equal to 0.077 in the model aquifer. In this case the model was assumed to represent a 8000 ft. section of a 100 ft. thick uniform aquifer.

The series of photographs shown in Figure 12a illustrates the wedge shape at various times during the overdraft period. The relatively greater toe velocity observed at first is due to the tendency of the interface to flatten out to a more horizontal position. During this period the interfacial movement near the top of the model is retarded while the toe velocity is accelerated. As the interface becomes more horizontal the toe velocity approaches the average velocity determined by the rate of withdrawal and the effective cross sectional area. As shown in Figure 11, this velocity is approached asymptotically.

Figure 12b illustrates the effect of injecting fresh water on top of the intruding saline water. The injection was q/T equal to 0.102. The overdraft rate and other conditions are the same as in the previous case. Here an injection process has been started after the top of the wedge has

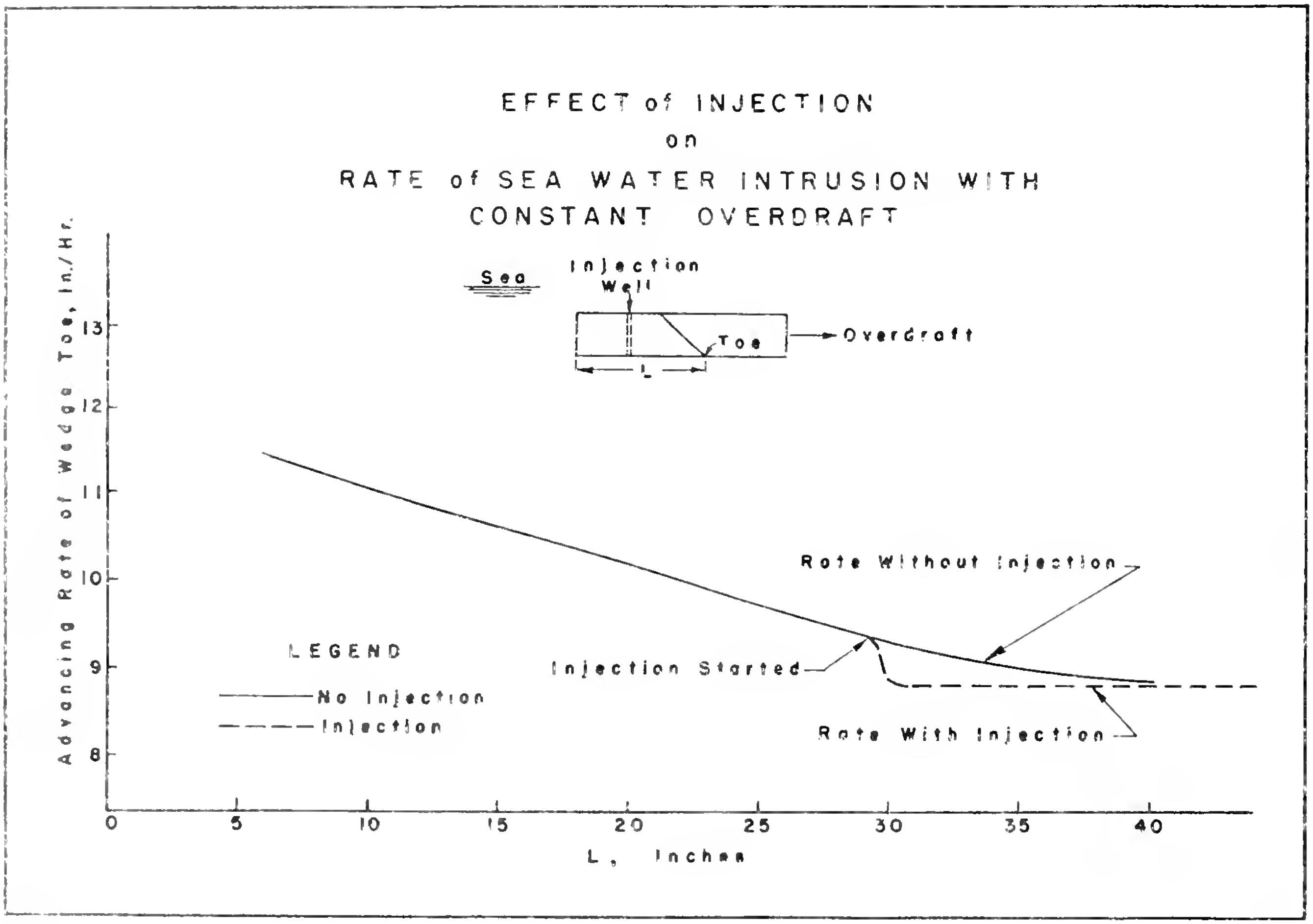
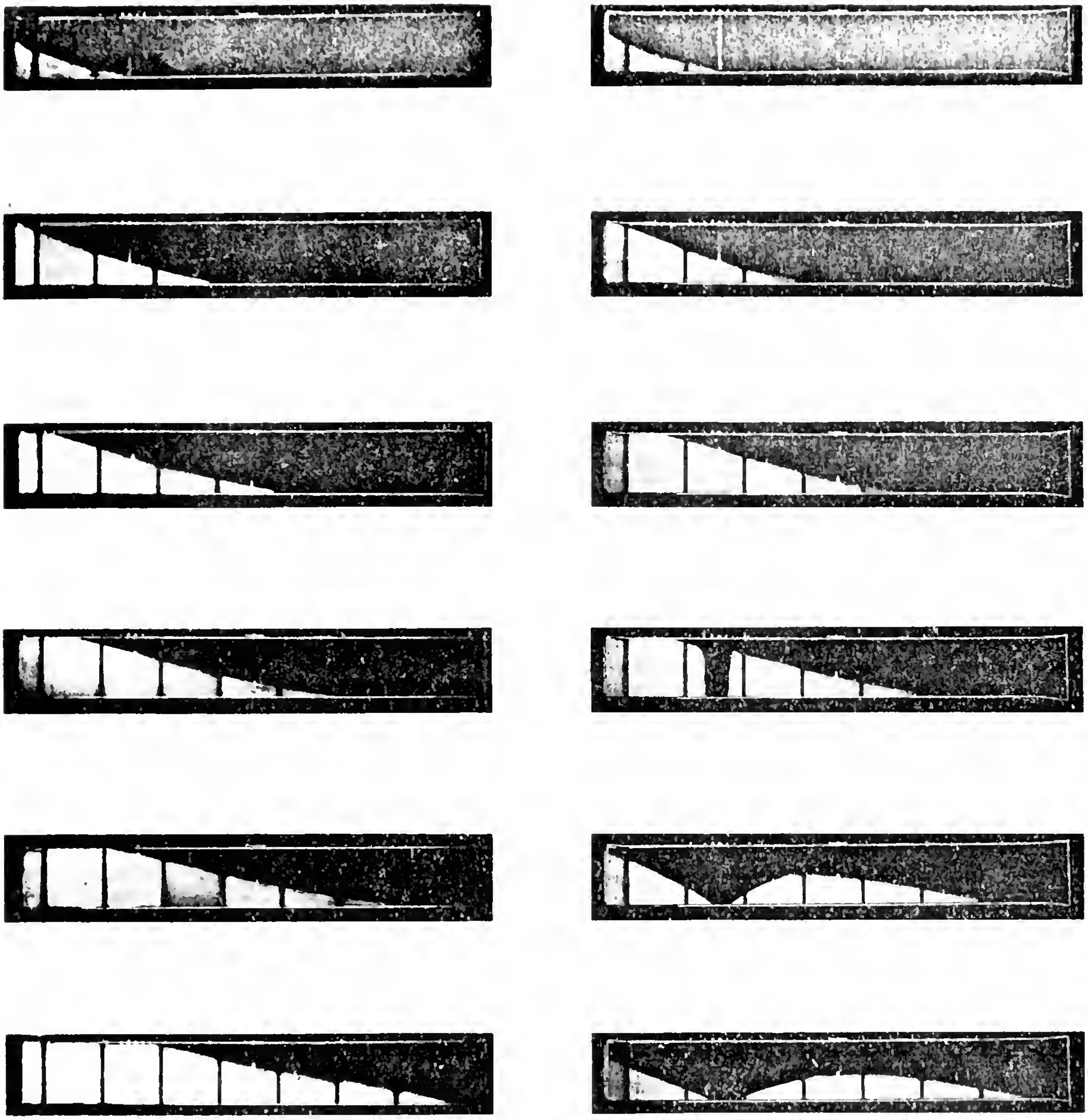


Figure 11 Effect of Injection on Rate of Sea Water Intrusion With Constant Overdraft.



(a) (b)

Figure 12. Behavior of Sea Water Wedge Under Overdraft Conditions, With and Without Injection of Fresh Water.

passed the injection well, at a rate equal to the overdraft rate plus an excess which provides for a seaward flow sufficient to stabilize the wedge toe twelve inches from the salt water chamber. The advance of the wedge toe is retarded slightly immediately after the start of injection, probably due to the fact that no additional salt water is available to maintain the slope of the interface. The section of the wedge which has been cut off forms a hump which flattens out as it continues to move inland. The rate of movement of the toe in this case is shown as the dashed line in Figure 11.

Overdraft and the Injection Rate

In a previous section the theory of potential fields was used to show that the successful operation of an injection well system requires that the injection rate be sufficient to satisfy the inland demand for water in addition to the flow required to maintain the sea water wedge seaward from the wells. This was based in part on the argument that no reduction in the inland flow should be expected as a result of raising the piezometric surface near the coast. The same conclusion can be reached through another line of reasoning.

The only storage capacity that a pressure aquifer has resides in the elasticity of the aquifer and of the water contained therein. Unlike the case of an unconfined aquifer, a lowering of the piezometric surface in a pressure aquifer does not indicate that an equivalent quantity of water has been withdrawn from storage. The effect of lowering the piezometric surface in a particular location is to induce inflow from regions of relatively higher pressure; if the aquifer is in open contact with the ocean floor the sea will serve as a recharging source as well as other contiguous areas. Unless the transmissibility of the aquifer near the coast is altered the coastal area will continue to supply its share of the recharging water demanded by the pumping zones. At present this supply is sea water in those ground water basins suffering from sea water intrusion; unless fresh water is supplied at a rate sufficient to substitute completely for the existing inflow of sea water, the remaining deficit will still be made up by sea water.

Model studies have completely confirmed these conditions; the injection of fresh water at rates less than the overdraft demand consistently results in the movement of salt water between the wells. The injection of fresh water at rates just equal to the overdraft, leaving none to flow seaward, has the same effect: the sea water wedge moves in and around the injection wells.

Injection Wells Partially Penetrating the Aquifer and Wells Injecting near the Aquifer Bottom

Several experiments were made to determine if any beneficial effect would result from injecting water near the bottom of the aquifer rather than through a well casing perforated throughout the aquifer depth. The fresh water chamber was closed off so that no water could enter or leave that end of the model. Fresh water was injected into the center well (at a simulated distance of 4000 ft. from the ocean outlet) which had had the slots in the upper half of the well closed off. Water issuing through the slots in the lower half of the well held the sea water wedge stationary at 2000 ft. when the appropriate flow rate was maintained. As the rate was doubled, the wedge moved to a new equilibrium position at 1000 ft. in exactly the same fashion it did when the injection process took place through all of the slots. Moreover, when the lower half of the well slots were covered, simulating a partially penetrating well, no difference in either the position or the movement of the wedge could be observed between this latter and the former two cases. The flow rate was the same in each case.

It is true that this action took place two well-spacings away from the line of injection wells. It may be argued that beneficial effects could have been observed closer to the wells. The only effect which would be beneficial for preventing intrusion would be the establishment of a vertical pressure gradient with the higher pressures near the bottom. A simple potential flow net analysis will show that it is impossible to maintain a vertical component of velocity (which must be associated with a vertical pressure gradient) of any magnitude beyond a horizontal distance toward an adjacent well equal to an aquifer thickness. Of course if there is a pronounced horizontal stratification with relatively impermeable strata existing within the aquifer, the injection of water into the lower permeable layers will create a higher pressure there than in overlying strata. In some instances such an aquifer structure will reduce the rate of seaward fresh water flow below that given in Equation 1. Such a structure would break down into a number of separate aquifers as the intervening layers became more impermeable. The inland flow rate demanded, however, would not be affected.

The Pumping Trough

Another proposed method for preventing sea water intrusion is to pump water from a line of wells near the coast, intercepting sea water before it can pass to the low pressure region

inland from the wells. An essential to the success of this method is the establishment of a seaward flow from the inland area towards the wells.

Preliminary model tests have shown that the sea water, upon reaching the wells, forms a wedge inland from them in a fashion similar to the way a wedge forms at an ocean outlet.

In the latter case a relationship has been shown to exist between the length of the intruded wedge and the rate of seaward fresh water flow. A somewhat analogous relationship seems to exist between the length of the sea water wedge which forms inland from the pumping wells and the rate of fresh water flow towards the wells. Here, however, there is some dependence on the sea water flow rate towards the well. Based on observations made during a model run in which 12 percent of the water pumped represented fresh water, the wedge may be expected to be on the order of 50 percent longer under these conditions than it would be at the ocean outlet with the same fresh water flow rate occurring in each instance.

This method would provide no recharge benefits, but the loss of fresh water need be no greater than that which would occur were the piezometric surface raised to the elevation required to hold back the wedge. In each case the fresh water loss would be proportional to the wedge length. In the pumping trough method the wedge would begin at the wells, but there would be no restriction on its maximum length (a longer length allows a smaller leakage of fresh water). The region seaward from the trough would be underlain by sea water. In the injection well method the wedge would begin at the ocean outlet and its maximum length would be fixed by the location of the wells.

DISCUSSION

The most obvious measurable quantity related to ground water phenomena is the height to which water will rise in a well. This height is directly proportional to the pressure within the aquifer. Emphasis on ground water elevations has in some instances tended to obscure the importance of ground water movement. The rate of ground water movement into a basin, for instance, must equal the rate of withdrawal plus the rate of addition to storage. No amount of adjustment to the pressures at the aquifer boundary can destroy this relationship, which is nothing else than an expression for the conservation of matter. An appreciation of this principle is essential to an understanding of sea water intrusion.

The height and slope of the piezometric surface around the boundary, of course, have a direct influence on the direction from which the inflow will come. In many ground water basins along the coast of California the quantity of fresh water now being pumped cannot be induced to flow from inland areas unless the piezometric surface is drawn down far below sea level. Where the aquifer is in direct contact with the ocean floor, however, recharge from the sea is possible. This source is unlimited, and has the advantage of being at a constant potential. Thus before the piezometric surface can be drawn down sufficiently to induce the required inflow of fresh water (which would be an impossible distance in many instances) a large inflow of sea water is induced. This sea water displaces fresh water from the coastal areas of the aquifer making it available to wells inland from the advancing front of intrusion.

If through some mechanism the height of the piezometric surface can be controlled along the coastal boundary of the aquifer, the inflow of sea water can be prevented. To accomplish this result, however, the elevation of the piezometric surface must be held above sea level because of the difference in density between fresh and sea water. The required elevation is shown to be a function of the sea water specific gravity in the section of this report dealing with fundamental considerations. As long as the fresh water piezometric surface is maintained at the proper elevation the sea water will not move inland, but will be established as a stationary intruded wedge. An illustration of such a wedge within a uniform aquifer is shown in Figure 10a. Associated with this wedge, however, is an inevitable seaward leakage of fresh water, the rate of which is inversely proportional to the length of the wedge from the ocean outlet. This means that if the wedge is allowed to move inland to a different equilibrium position the seaward leakage will be reduced. A method for determining the amount of this seaward leakage for a uniform aquifer is presented in the section on fundamental considerations, and modifications of the theory to allow an estimation for non-uniform aquifers is presented in a later section on experimental results. The important thing here is that in order to hold the intruded sea water wedge at equilibrium a definite amount of seaward leakage of fresh water must be allowed.

The only presently conceived method for raising the piezometric surface in a pressure aquifer is the injection of fresh water into it through wells. Both theoretical considerations and the results of model tests have shown what the injection rate must be to assure the success of such a method. It must equal not only the seaward flow rate required to stabilize the position of the sea water wedge, but also the

entire overdraft rate which has originally caused the sea water intrusion. Effectively, the injected fresh water replaces the sea water which would otherwise flow inland. Since the quantity of water which must be injected to insure the success of the injection well process is equal to the overdraft rate, the question arises as to whether it would not be more feasible to use this water directly on the surface rather than to subject it to the troublesome and expensive procedure of first injecting it into the ground and then pumping it back out again. The use of reclaimed water, however, may eliminate a part of the expense involved in the injection process. If reclaimed water is to be used, an economic analysis should be made of the relative cost of (1) treating it to an extent which would allow its continuous injection into the aquifer, plus the cost of injection and pumping; and (2) the cost of treating it to the extent necessary to make it directly usable for industrial purposes on the surface, plus the cost of distribution.

The pumping trough deserves attention as a means of preventing the inflow of sea water. This method would provide no recharge benefits, but the seaward leakage of fresh water (to the wells in this case) need be no greater than in other methods, as described in a previous section. The extraction rate would have to be adjusted so that this amount of fresh water is pumped along with the intercepted sea water. Ground water levels would take a further large drop in many instances, since recharge from the sea would be prevented and all of the water would have to come from inland areas. The rate of inflow from inland areas may not be enough to keep aquifers which are now under pressure completely full of fresh water, and this would operate to bring the pumping rate into line with the fresh water recharge rate. This procedure would not be popular, but it is suggested that no plan for bringing the pumping rate into line with the natural fresh water recharge rate is likely to be popular.

If an impermeable cut-off wall were to be constructed along the coast these same conditions would obtain, except, of course, that no seaward leakage would need to take place. The amount of seaward leakage required in many instances, however, is not a large percentage of the natural fresh water inflow.

THE INVESTIGATION

II. IMPERMEABLE CUT-OFF WALLS

THE CUT-OFF WALL

In the long run, the chief value of an impermeable cut-off wall between fresh and saline waters in an aquifer would be that underground fresh water storage could be developed below sea level. This would create greater capacity for salvage of fresh water which might otherwise escape to the ocean during wet years. Under these conditions the cut-off wall must prevent the intrusion of sea water which would render significant parts of the underground supply useless.

Figure 13 shows a vertical section through an aquifer and cut-off wall in a plane perpendicular to the sea coast. For the purpose of estimating the degree of impermeability required for such a cut-off wall, the fresh water

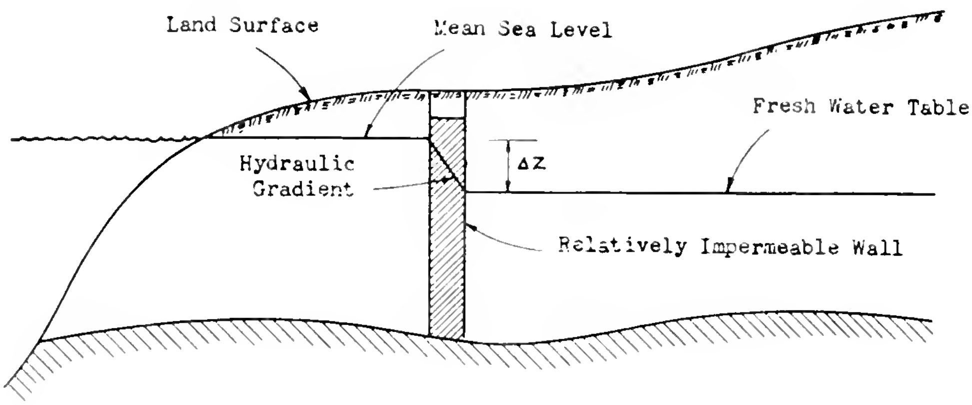


Figure 13. Section Through Aquifer and Cut-off Wall.

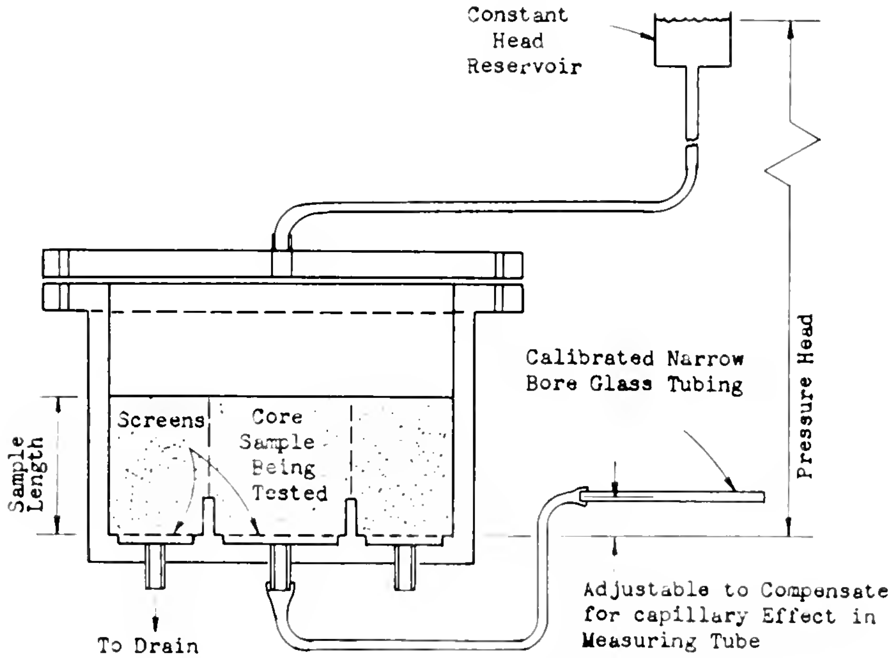


Figure 14. Detail of Permeability Test Cylinder.

level on the landward side of the barrier has been taken to be thirty feet below sea level, and the cut-off wall to be three feet thick. Under such conditions the hydraulic gradient through the barrier would be 1000%. The estimated dimensions of a barrier proposed at San Luis Rey are about 150 feet deep and about 1000 feet wide. The amount of sea water which might be expected to flow through such a barrier, under these conditions, can be calculated from the permeability coefficient found for the backfill material. Assuming a value of .01 gallons per square foot per day per 100% hydraulic gradient and the area to be 150,000 square feet, the salt water flow through the barrier would be 15,000 gallons per day. This is equivalent to 17 acre-feet per year, and 17 acre-feet of sea water will pollute a much larger volume of fresh water beyond the upper limit of safe use for irrigation. Pumping immediately inland from the barrier could intercept this salt water leakage. At the start of operations it might also be desirable to pump and waste unusable groundwater from the landward side of the barrier for whatever period of time necessary to eliminate intruded sea water trapped behind the cut-off wall.

NATURE AND USE OF BENTONITE

General Properties of Bentonite Clay

Bentonite is a naturally occurring montmorillonite clay which possesses several properties which have recommended it as a sealing agent for reservoirs and as an additive to drilling fluids in oil well construction. In contact with fresh water, it expands to about ten times its dry volume, forming a gelatinous mass. If it can be applied in such a fashion that impregnation occurs before the bentonite achieves its maximum swell, it provides a good seal in porous formations. As an illustration, dry bentonite may be disced into the surface soil at a reservoir site; upon subsequent flooding it swells to convert the upper layer of soil into a jellylike impermeable blanket. A gelatinous mass can be diluted to form a slurry; when such a slurry is forced against a filtering medium, water is expressed and the gelatinous characteristic is regenerated. As utilized in drilling operations, such a slurry tends to deposit an impermeable layer or "cake" of this gelatinous material next to the well wall upon which hydrostatic pressure can act in supporting the wall against cave-ins.

Sealing Mechanism of Bentonite in Drilling Muds and in Cut-off Walls

In well drilling operations a virtually unlimited supply of slurry is available to plug porous formations. In addition, fine sand and bit cuttings become suspended in the slurry, and should a leak develop, these tend to clog th hole and provide a base upon which a cake a be formed. Should the interstices of the po formation be too large to trap the naturally o curring suspended fines, additional fibrous m terial can be added to the slurry.

During the excavation of a trench which s kept full of bentonite slurry, a relatively im permeable cake should build up wherever e is a tendency to lose fluid. However, such cake is quite thin and may not remain in p c during backfilling operations. If its contribu tion to the cut-off wall is disregarded, the a fill moisture itself must be depended upon o provide a seal.

That the sealing action of bentonite slurry used as an additive to a backfill mixture c n be compared to its action in sealing an ope cavation is evident from the results of the o lowing experiments:

A uniform coarse sand (passing U.S. sive No. 16, but retained on No. 30) was packed n four-inch layer within a two-inch plastic t e and saturated with a 15 centipoise slurry e scribed by oil well drillers as an ideal pump able and workable slurry). After waiting n minutes to allow any initial shear to develp, hydraulic pressure gradient was applied; d these conditions the slurry was invariably flushed out under less than 100 percent hy draulic gradient. The experiment was repa except that the slurry-saturated layer was sandwiched between two water-saturated ly of the same sand; the results were the sare.

Apparently the fines which are found su pended in oil drilling muds are essential t i sealing action; therefore, the next step wa t incorporate fine material, passing a 200 m s screen, into a backfill mixture of well grae fine and medium sands. Upon the applicatior a moderate hydraulic gradient the fine mater tended to retain the bentonite within the mixt allowing water to pass through. The actic c this fine material in initiating a seal in a op trench during excavation and as an additiv t backfill mixture illustrates the difference n two applications: in a slurry filled hole o trench, slurry and suspended fines convere any leak which may develop; in a cut-off wl the fines must be provided in sufficient amo at every point, for in a non-fluid mass no at movement from adjacent areas could be ep to make up a deficiency at a weak spot.

EXPERIMENTAL PROCEDURES AND RE-SULTS

Permeability Coefficients

Laboratory equipment was designed to deter mine the permeability coefficient of backfill

naterials under the same hydraulic gradient ex-
pected in the prototype wall. A cut-away view of he permeability testing cylinder used is shown n Figure 14 (Page 29).

The design employed is one which corrects or any sidewall leakage. The same back pres- sure was maintained at the center screen and at he annular screen in order that flow down hrough the total cross section would be in paral- el streamlines. Any sidewall leakage was hereby isolated from the central core being ested. The flow through the core was deter- nined by measuring the rate of travel of the neniscus in the calibrated narrow-core glass ube. Sufficient hydrostatic pressure was ap- lied to induce a 1000% hydraulic gradient with- n each sample, and permeability measurements vere made over relatively long periods of time, p to two weeks in some instances, to evaluate he effect of continued flow on the properties of he material.

hin Bentonite Slurry as an Additive to Back- ill Mixtures

In oil well drilling practice a 15 centipoise lurry is considered an ideal pumpable and orkable drilling fluid. Accordingly, with the ope that the bentonite slurry used to maintain ne walls of a trench during excavation could lso be used as an additive to the backfill mix- ire, several experiments were conducted to de- ermine the permeability of different mixtures f sands and silts with this type slurry. Three and-silt mixtures were used in this and subse- uent tests; the size distribution is shown in able I. Coarse sand and gravel were not in- luded in any of the backfill aggregates tested; ne permeability coefficient of mixtures contain- ng reasonable additions of such coarse mate- ial should not be appreciably different from the ermeability coefficient of the fine material lone.

TABLE I
Particle Size Distribution of Sands Used with Bentonite in Permeability Studies

Classification	Size Limits, mm	Aggregate A	Aggregate B	Aggregate C
Medium Sand	0.6 to 0.2	68.2%	58.8%	14.8%
Find Sand	0.2 to 0.06	20.4	28.1	51.0
Silt	0.06 to 0.002	10.2	15.3	30.6
Clay	less than 0.002	1.2	1.8	3.6

In accordance with a procedure used by Mr. eorge Cummer in his tests on samples of pro- osed backfill material for the Pasco, Washing- on, cut-off trenches,* only sufficient 15 centi- oise slurry was added to aggregate A to fill the calculated voids. Such a mixture was very stiff, and had to be placed in the test cylinders by the spoonful, where it was carefully tamped. In terms of the concrete consistency test, which was made on most of the subsequent samples, its slump was zero. When additional slurry was added to other sand samples of the same composition, a workable mix was developed.

The permeability to fresh water, as meas- ured after an eight hour period of flow, is shown in Table II for samples containing different amounts of excess slurry. "Excess slurry," as used in this report, refers to the volume of slurry added to a sand-silt mixture in excess of that required to fill the calculated voids. The void volume for each aggregate was determined after vibration to maximum density; the quan- tity of excess slurry is expressed as a percent- age of this volume.

TABLE II
Permeability of Various Aggregates Mixed with 15 Centipoise Slurry Measured at 1000% Hydraulic Gradient

Aggregate	Excess slurry added	Permeability at end of 8 hrs. gal/sq.ft./day/100% gradient
A	0%	1.0
A	10%	2.9
A	20%	5.8
B	To provide 4" slump*	5.0

* In concrete-consistency test

Although the permeability coefficients re- ported in Table II are on the order of one- thousandth of the value for a typical aquifer material, this fact does not indicate that a cut- off wall of such material would allow only a similar proportion of sea water intrusion com- pared to the section of aquifer it may replace. In practice a large hydraulic gradient would build up across the barrier because of its rela- tive resistance to flow; depending on individual conditions, such an induced gradient could largely nullify the effectiveness of the barrier unless its permeability coefficient was relative- ly quite low. It appears that a satisfactory barrier can be constructed of backfill material having a permeability coefficient no greater than 0.01 gal/sq. ft./day/100% gradient. No backfill mixture incorporating 15 centipoise slurry approached this value.

Thick Bentonite Slurries as Additives to Back- fill Mixtures

In an attempt to achieve a more impermeable backfill mixture, slurries containing a larger percentage of bentonite were prepared and tested in conjunction with the same sand-silt aggregates. These slurries, together with the 15 centipoise slurry, are described in Table III.

Figure 15 illustrates the type of data obtained, it is a plot of permeability versus time for back-

* Unpublished report to Cronese Products Co., ated January 3, 1951.

TABLE III

Bentonite Slurries Tested as Additives to Backfill Mixtures

Slurry designation	Percentage of oven dry bentonite in slurry	Viscosity, stormer centipoise value
"15 centipoise"	4.38	15.0
"6%"	6.00	66
"7%"	7.00	285*
"8%"	8.00	515*

* grams to drive stormer at 600 r.p.m.

fill mixture of aggregate "A" and "7%" slurry. The data for three identical runs, all determined for a core length of 10 centimeters, are shown together with similar data for a core length of 30 centimeters. The experimental technique employed was identical for each of the runs, and measurement errors were negligible compared with the variability between samples. The conclusion is that the variation is real, and that it reflects a difference in the way the material within the sample becomes rearranged upon the application of a hydraulic gradient. Such variations should become smaller as the length of the sample is increased. The values obtained for the sample with a 30 centimeter core length for instance, lie entirely within the range of values obtained for the 10 centimeter core lengths, and are close to the value obtained by averaging those found for the shorter cores. The 10 centimeter cores were used for most of the other tests with the justification that more samples using less material could be analyzed with this procedure. In addition, the variability of the values obtained provides a measure of the reliability of the projected cut-off wall's

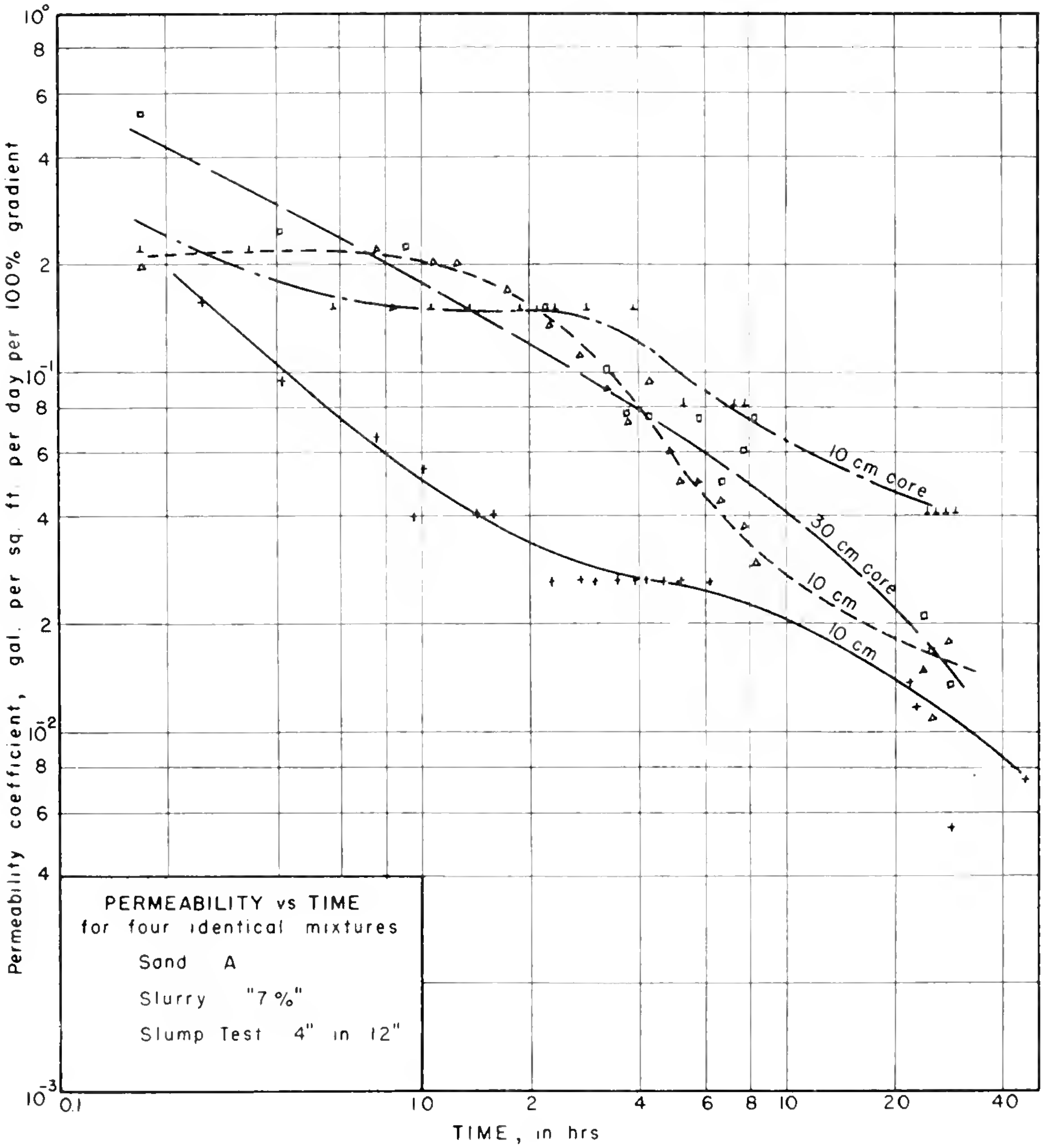


Figure 15. Change in Permeability Coefficient with Time.

impermeability, since the maximum variability expected in the prototype should be displayed in these cores.

The downward trend of permeability coefficient values with time should be noted from Figure 15. This relationship is a general one; however, the rearrangement of material within the sample can take place to a limited extent only, and it is unlikely that the permeability coefficient would decrease indefinitely. In some instances, particularly when a large excess of slurry was employed, an initial decrease of the permeability coefficient value was followed as much as 24 hours later by an abrupt increase as piping through the sample developed.

An illustration of this phenomena as well as a demonstration of the effect of excess slurry on the permeability coefficient, is provided in Figure 16. An earlier reference has been made to the fact that a backfill mixture containing only sufficient slurry to fill its voids is unworkable. Although it has a favorably low value of permeability coefficient, there seems to be no practical way to tamp such a mixture into a trench already filled with slurry without inadvertently incorporating additional slurry within it. Also, one of the proposed methods of backfilling, in which backfill material would be deposited near the surface and sloughed down a slope of already-deposited material toward the bottom of a slurry-filled trench, assumes a semi-plastic mass. As shown in Figure 16, the incorporation of excess slurry, either inadvertently or purposely, compromises the desirable properties of the backfill material. Once a moderate amount of excess slurry has been incorporated, it seems unlikely that it would segregate from the backfill mixture for the same reason that portland cement slurry does not segregate from normal mass concrete. In view of the above considerations, particular attention was paid to mixtures with moderate workability.

Data for backfill mixtures having a consistency, in terms of the concrete slump test, of from three to four inches in twelve have been compiled; average values of the permeability coefficient for aggregates A, B, and C are tabulated in Table IV. These values should not be considered precise, but do represent a reasonable order of magnitude for the permeability at the end of twenty-four hours.

In general the permeability coefficient decreases with an increase in the amount of silt in the mixture. The value obtained for different strengths of bentonite slurry employed, however, seems to reach a minimum for the 7 percent slurry. The explanation is that the 8 percent slurry was so thick that a large excess had to be added to achieve a reasonable workability; the adverse effect of the excess slurry apparently was greater than the favorable effect of its increased stiffness. When a smaller excess of the 8 percent slurry was added, a corresponding decrease in the permeability was noted, as shown in the last line of Table IV.

TABLE IV
Permeability Coefficients, gal/sq.ft./day for Workable Mixtures

Slurry Designation	Aggregate (See Table I) A	B	C
6	pipe develops	.01	.007
7	.020	.004	.007
8	-	.02	.004
8	.015*	.002*	-

* Mixture had only 2" slump.

Bentonite with Other Additives

Portland Cement

It was thought that small additions of portland cement might, upon setting up, form a network of intermeshed calcium aluminium silica crystals within the mixture which, though they would not impart very much strength to the aggregate, might form a base upon which the bentonite slurry could form an impermeable "cake." The cement was added in amounts of 5 and 7-1/2 percent to aggregate "A," and sufficient 15 centipoise bentonite slurry was then added to form a workable mixture having a three to four inch slump from twelve inches. Either because a cake did not form within the mixture or because it was insufficiently continuous, the permeability coefficients of these mixtures were not appreciably smaller than those of mixtures which did not contain the cement. Mixtures incorporating larger percentages of cement were not tested, since it appeared that they would not be economical.

Asphalt Emulsions

A slurry-asphalt emulsion mixture was prepared incorporating 20% by weight of the emulsion and added to aggregate "A," as in the experiment described above. This mixture developed a permeability coefficient of only .05 gal. per square foot per day per 100 percent hydraulic gradient and did not tend to set up to an unworkable mass.

DISCUSSION

Compaction of the Backfill Mixture in Place

Since the desirable properties of backfill mixtures are compromised by the incorporation of excess slurry over that required to fill

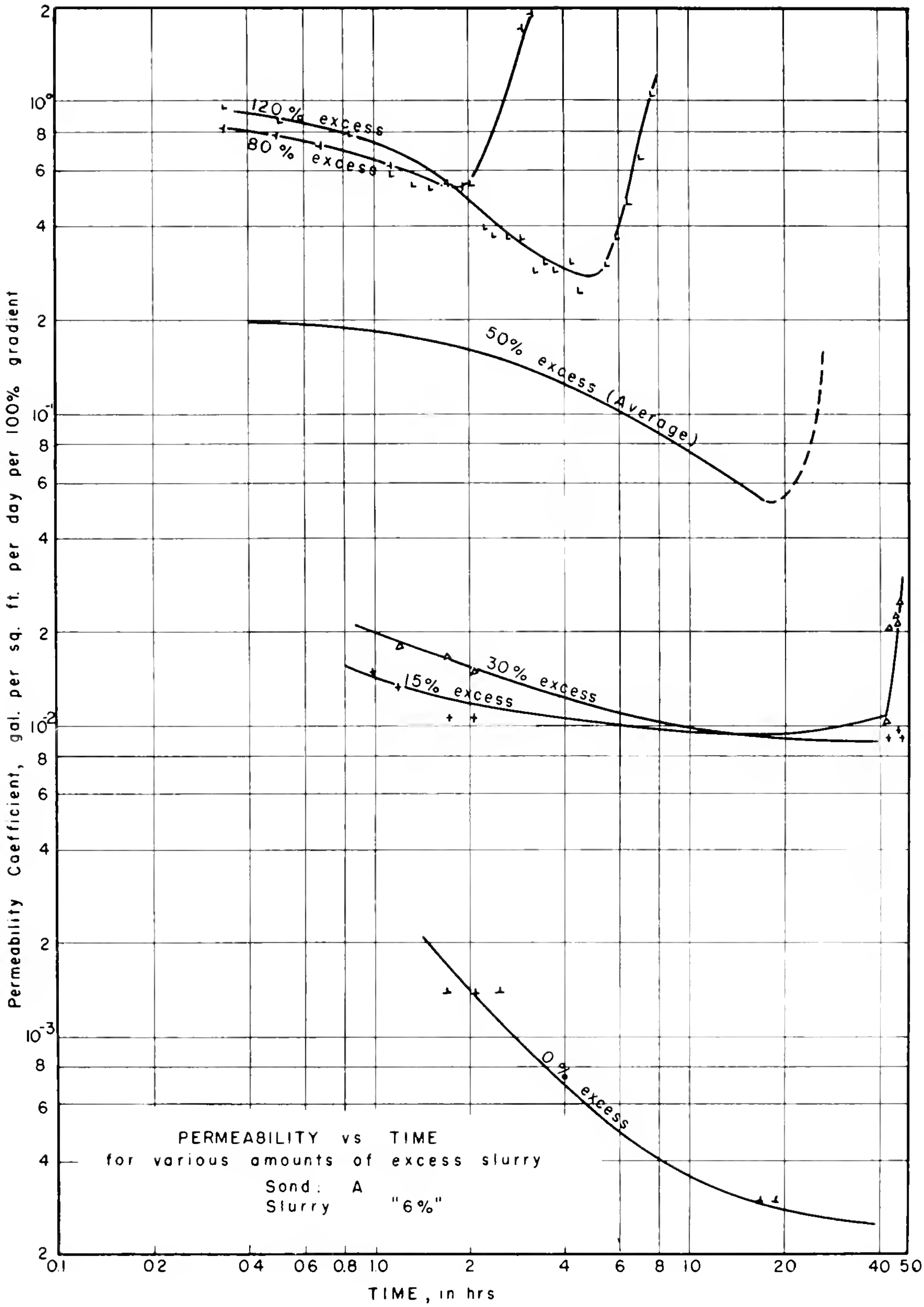


Figure 16. Effect of Excess Slurry on Permeability Coefficient.

the voids, some question may arise as to whether the extreme pressure in a prototype cut-off wall may not be effective in compacting the mixture after placement, squeezing the excess slurry into the surrounding formations, thus, although excess slurry must be added in the placement process, it may be argued that the effective properties of the backfill mixture in place would be more similar to those of the core samples containing no excess slurry over that required to fill the voids.

In analyzing the above possibility a distinction must be drawn between hydrostatic pressure and the pressure exerted by the aggregate. The former type of pressure exists in a plastic mass, such as concrete or an aggregate-slurry mixture, and is exerted equally in all directions. During the excavation of a slurry filled trench, escape of the slurry is prevented by the formation of a "cake" on the trench walls, upon which the hydrostatic pressure acts to hold them up. Whether this "cake" will be undisturbed throughout the area of the wall during backfilling operations is problematical. Where it is maintained over a sufficient area, hydrostatic pressure within the backfill mixture may be higher than the water pressure in the surrounding formations. Where gaps in the wall "cake" occur the hydrostatic pressures in the mixture and in the surrounding formation will tend to equalize, assuming a slurry as thin as 15 centipoise.

The excess slurry cannot be squeezed out by hydrostatic pressure, however. It can be pressed out only by pressure exerted by the aggregate particles themselves, from grain to grain. There is no doubt that complete compaction would occur eventually, and that the aggregate grains would settle down upon each other, pressing the excess slurry out. There is a strong possibility, however, that bridging may occur in many places in a trench of the proportions envisioned for cut-off walls designed to prevent sea water intrusion. This bridging phenomena was observed in the laboratory when an attempt was made to compact backfill mixtures in smooth Lucite cylinders two inches in diameter. Excess slurry could be squeezed out of the mixture in about two inches of the cylinder (a length equal to its diameter) using hand pressure on a perforated piston, but as the piston gathered more compacted material ahead of itself, the force required to continue its motion rapidly increased. Even if bridging phenomena protected only ten percent of the cut-off wall from compaction, the properties of the mixture as placed (containing excess slurry) would have to be such as to prevent a break through at these points.

Accordingly emphasis in the testing procedures has been on the properties of workable mixtures, and it is recommended that no reliance be placed upon the possibility that compaction will be effective throughout the cut-off wall.

Comparison of Laboratory and Field Mixing Technique

Since the laboratory measurements of permeability were made on relatively small samples, every precaution was taken to insure uniformity within the mixture. Whenever bentonite was to be added to a mixture, it was first blended in water to a homogeneous paste or slurry and added to dry aggregates. This has led to terminology, such as "7% slurry," which is convenient for laboratory use. Actually it is the ratio of bentonite to water which must be controlled, and any field mixing technique which will insure that a proper ratio is maintained should be satisfactory. A promising procedure might be to incorporate a portion of the bentonite in granular form; a backfill mixture thus fortified would have better workability than one in which the bentonite is completely hydrated before placement. Conversely, a lower and more favorable water-bentonite ratio could be maintained for the same workability. Preliminary investigation indicates that several hours to a day elapse before granular bentonite achieves its maximum swell when placed in contact with fresh water.

When fully hydrated bentonite slurry is added with or without small amounts of asphalt emulsion, the mixture tends to retain its workability and does not set up to a stiff mass.

Economic Comparison of Backfill Mixtures

Several technically feasible backfill mixtures have been developed incorporating various additives. The specific performance of these mixtures is dependent on the size and gradation of the aggregate available. Within the limits indicated, Table V gives an economic comparison of several types of mixtures with satisfactory low permeability coefficients.

TABLE V

Comparison of Cost of Several Types of Mixtures

Admixture	Approximate permeability coefficient. Gal/sq.ft./day/100% hydraulic gradient	Effect of sea water flow on permeability	Approximate cost of admixture, per cu. yard of backfill (1)
1/3 clay by wt. to equal parts sand and gravel	.01	small	\$4 [b]
Thin, bentonite slurry containing 20% asphalt emulsion (2)	.05	small	\$4 [a,c]
Wyoming bentonite, added uniformly to equal 7% of water content of mixture (2,3)	.001	increases permeability by a factor of approx. 10	\$.75 [a]
Clay puddle	.01	estimated to be small	\$12 [b]
Chrome-lignin	unknown	unknown	\$4 - \$11 [d]

(1) Does not include labor and assumes that sand, silt and gravel are free.

(2) Aggregate must contain 10-15% silt, 30-50% sand.

(3) Only sufficient water should be added to achieve workability.

(a) Based on \$30 a ton for Wyoming bentonite.

(b) Based on \$10 a ton importing cost for clay.

(c) Based on \$50 a ton for asphalt emulsion.

(d) Based on patent holder's estimate.

SUMMARY AND CONCLUSIONS

Thirteen ground water basins along the California coast have already been damaged and others are being threatened by the encroachment of saline waters from the ocean or inland bays. To meet this threat, the State Water Resources Board began conducting an extensive investigation of ways of controlling or preventing sea water intrusion. As a part of that investigation, the University of California on January 1, 1952 undertook a program of research on sea water intrusion into fresh ground water sources and methods of its prevention, under Standard Service Agreement No. 3 SA-423 between the Regents of the University and the State Water Resources Board. On December 1' 1952 the closing date of this agreement was extended to July 31, 1953. The work of the University was confined to the determination of theoretical relationships and to laboratory research. This report covers work done on the basic parameters of sea water intrusion, on the hydraulics of injection wells, and on the construction of impermeable cut-off walls.

It is a well known fact that the fresh water piezometric surface must be maintained above sea level in order to prevent the encroachment of saline water into an aquifer which has direct access to the sea. In the course of the studies herein reported theoretical relationships were derived which give the distance above sea level to which the piezometric surface must be raised in order to halt the intrusion, as well as the amount of seaward fresh water leakage to be expected under these conditions for an aquifer of known characteristics. Also a theory was developed which shows that a line of injection required for aquifers under different hydraulic conditions can be predicted.

In order to confirm this and other theories, a model pressure aquifer six inches high, three inches thick, and four feet long was constructed of Lucite. It was packed with a clean quartz sand, and provision was made for introducing fresh and salt water through either of two end chambers, or through either of two model injection wells. The salt water was distinguished from the fresh by fluorescent dyes, and the interface could be observed both in a plane through an injection well and in a plane midway between injection wells. Experimentally the sea water was observed to enter the aquifer as a wedge along the lower boundary and to move to an equilibrium position which depended on the seaward rate of fresh water flow. The position and movement of this intruded sea water wedge were recorded for various rates of seaward fresh water flow, and also for various rates of overdraft. Later the effect of injecting fresh water into the zone of intrusion and into the aquifer inland from the sea water interface were observed. Preliminary studies were also made of a pumping trough, using the model wells to extract water from the aquifer in amounts sufficient to intercept all of the intruding sea water. Incidental data, such as the relative effect of diffusion that may be expected, were also obtained. Throughout many of the experimental runs an automatic lapse-time motion picture camera was used to obtain a continuous record of the position of the interface.

The principal conclusions which may be drawn from the experimental data obtained in studies of sea water intrusion include the following:

1. In order to prevent sea water from entering an aquifer which has direct access to the sea, the fresh water piezometric surface must be held above sea level a distance equal to (S-1) times the distance below sea level to the lowest pumping zone which must be protected, where S is the specific gravity of the ocean or inland bay water.

2. For aquifers of finite thickness, the maintaining of a fresh water surface above sea level will result in a seaward leakage of fresh water in the upper portions of the aquifer. A sea water wedge will form in the lower portion, extending inland from the ocean outlet a distance which is inversely proportional to the fresh water flow rate. For a uniform aquifer this seaward leakage may be determined from formulas given in this report; for non-uniform aquifers methods are given for its estimation.

3. The relationship between the equilibrium wedge length and the rate of seaward flow of fresh water is independent of the distance of the aquifer below sea level.

4. There is no marked change in the shape of the fresh water-sea water interface at the beginning of an overdraft period. From any initial position the interface moves inland at a rate determined by the rate of fresh and salt water movement. In a uniform aquifer, the wedge tends to flatten out and the toe tends to move somewhat faster than the interface as a whole due to the greater density of the salt water. In most prototype aquifers, however, the uncertainties of non-uniformity within the aquifer make it impossible to generalize on the rate of intrusion, since the sea water may enter the more permeable portions and travel relatively rapidly within them.

5. If fresh water can be injected into the aquifer at a sufficient rate, the piezometric surface can be maintained at the required height above sea level in a region along the coast. The spacing of the wells is of little importance, except that the toe of the intruded wedge should be held at least half a well spacing seaward from the centerline of the wells. The seaward fresh water leakage will be related to the length of the sea water wedge in the same way as before. However, unless the inland demand for fresh water is reduced, the injection rate must equal not only the leakage rate, but also the entire overdraft rate which has originally caused the intrusion.

6. If fresh water is injected on top of the wedge at a rate sufficient to halt intrusion, the portion of the wedge extending inland from the wells will be cut off. It continues to move inland depending on the hydraulic gradient existing there, but its rate of travel will be no greater, and actually will be somewhat less, than the rate of travel of the entire wedge under the same overdraft conditions.

7. Unless there is a pronounced impediment to vertical flow within the aquifer the injection of water near the bottom of the aquifer provides no benefits in the form of a reduced leakage rate.

8. Intruding sea water can be intercepted by a line of pumping wells, which would form a "pumping trough" near the coast. There would be no recharge benefits from this plan, but under proper operation the seaward leakage of fresh water need be no greater than in the injection process.

Bentonite slurry as an additive to backfill mixtures did not meet original expectations. The addition of very fine material to the backfill mixture and the use of rather stiff bentonite slurry markedly reduced its permeability. However, the sealing action of bentonite slurry in such an aggregate was of a different order of magnitude than that expected on the basis of its properties before incorporation in the mix. The results of the permeability tests as well as other evidence show that the gel strength of bentonite slurry is impaired by the presence of closely packed fine materials. A possible explanation is that the electro-kinetic potential between the fine grains and the slurry adversely affects the latter's ability to form a gel.

From the results of laboratory tests with impermeable cut-off walls the following conclusions are justified:

1. Bentonite slurry of a consistency ordinarily used in drilling operations is not a satisfactory additive to backfill mixtures containing up to 30% silt.

2. Stiffer slurries, containing from seven to eight percent dry bentonite, promote a satisfactorily low permeability coefficient in well-graded backfill mixtures containing from 15 to 30% silt.

3. It is unlikely that excavation equipment can operate in these thick slurries; even tetrasodium pyrophosphate was unable to reduce their viscosity appreciably. Unless some material is found that will act in conjunction with the thinner trench slurry to form a satisfactory additive, a separate source of slurry must be used with the backfill mixture.

4. Only enough slurry should be added to the backfill mixture to obtain minimum workability; any additional amount raises its permeability and contributes to the danger of piping. For this reason a method of backfilling should be selected which will allow a minimum amount of water or trench slurry to be incorporated in the mix during placement.

5. Preliminary data indicate that the permeability to sea water of a mixture incorporating bentonite slurry is roughly ten times its permeability to fresh water.

6. No mixture was found which provides a complete seal. The residual leakage may be determined from the permeability value for the barrier and from a knowledge of ground water elevations obtaining on each side. The resulting damage would depend on the diluting effect of fresh water flow into the basin and on the amount of water exported.

APPENDIX I

DEVELOPMENT OF THE EQUATION FOR THE LENGTH OF THE INTRUDED WEDGE

In Figure 1 (Page 12), it is assumed that the intruded wedge is stationary and it is proposed to calculate the seaward flow of fresh water in the region above it. This assumption implies that there are no pressure gradients within the wedge and that the pressure within it is everywhere the same as at an equal depth in the ocean.

The exact calculation of the fresh water flow rate requires the solution of Laplace's equation for the fresh water potential distribution in the region occupied by fresh wa -, subject to appropriate boundary conditions. At the toe of the wedge the fresh water potential, $\emptyset$, may be assumed as constant within a vertical plane. If H is the vertical distance of the lower aquifer boundary below sea level, the pressure at the toe of the wedge must be $w_S H$, where w_S is the unit weight of sea water. This pressure must be the same on each side of the interface, so that the fresh water pressure there is also $w_S H$. The potential energy of the fresh water is, relative to sea level,

$$\emptyset = \frac{p}{w_f} - z \qquad \text{Eq. 1-1}$$

where w_f is the unit weight of the fresh water and p is the pressure at the distance z below sea level. Referring to Figure 1-1 below, the value of the potential along A-B is then

$$\frac{w_s}{w_f} H - H = (S-1)\, H,$$

where S is the specific gravity of the sea water.

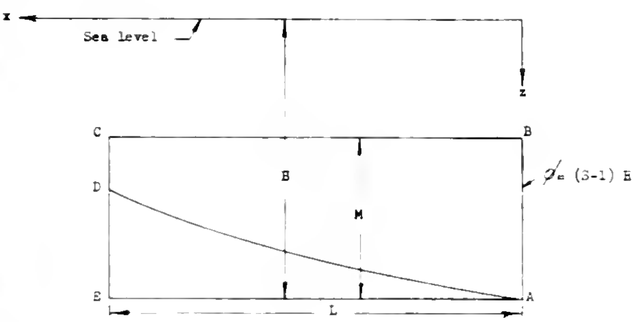


Figure 1-1

At each point along the fresh water - sea water interface, CDA, the potential is thus (S-1) z. Two remaining conditions are that BC and AD must be streamlines. The boundary AD corresponds to the free surface in the comparable flow pattern of seepage through an earth dam on an impermeable foundation. Such gravity flow systems can be solved in an exact fashion only by the involved method of hodographs (41).

However, by applying reasonable simplifications to the boundary conditions, approximate solutions can be obtained with less difficulty. Thus the potential at E is the same as at A, and even without the imposition of the streamline AD no flow would be expected between these two points. Muskat (42) has simplified the above conditions by removing the free surface AD, and assuming that the potential along CE is (in our terminology) (S-1)z. (Muskat (42) indicates an error of about one percent between the exact and approximate solutions.) The solution of the Laplace equation,

$$\frac{\partial^2 \emptyset}{\partial x^2} + \frac{\partial^2 \emptyset}{\partial z^2} = 0,$$

may be expressed as

$$\emptyset = (S-1)H - \frac{1}{2}(S-1)\frac{Mx}{L} + \frac{2(S-1)M}{\pi^2}\sum_{1}^{\infty}\frac{(-1)^n - 1}{n^2}\cos\frac{n\pi(z-H+M)}{M}\,\frac{\sinh\frac{n\pi x}{M}}{\sinh\frac{n\pi L}{M}} \qquad \text{Eq. 1-2}$$

The total flow through AB may be computed by integrating the flow through each element dz between the limits H-M and H. The flow through each element by Darcy's law is:

$$dq = -K\left(\frac{\partial \emptyset}{\partial x}\right)_{x=0} dz \qquad \text{Eq. 1-3}$$

The total flow is then

$$q_{AB} = -K \int_{H-M}^{H} \left(\frac{\partial \emptyset}{\partial x}\right)_{x=0} dz \qquad \text{Eq. 1-4}$$

In the integration each term of the summation drops out because of $\cos \frac{n\pi(z-H+M)}{M}$ and we left with:

$$q_{AB} = \frac{K}{2}(S-1)\frac{M^2}{L}$$

$$= \frac{1}{2}(S-1)\frac{M}{L}\, T \qquad \text{Eq. 1-5}$$

where MK = T, the aquifer transmissibility.

APPENDIX II

THE CONSTRUCTION OF EQUIPOTENTIAL CONTOURS ASSOCIATED WITH INJECTION WELLS

The potential distribution due to the steady state flow of water through a uniform confined aquifer must satisfy the Laplace equation; if vertical velocity components can be neglected, the distribution is essentially two-dimensional. Assuming the x, y cartesian coordinate system shown in Figure 2 of the test, the equation to be satisfied is:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad \text{Eq. II-1}$$

in which h is the height of the piezometric surface.

The potential distribution due to a finite number of injection wells operating between constricting lateral boundaries is in all respects representative of that produced by an infinite line of wells in the infinite x, y plane. This distribution is given by:

$$h = h_o - \frac{Q}{4\pi T} \log_e \left[\cosh \frac{2\pi y}{a} - \cos \frac{2\pi x}{a} \right] \qquad \text{Eq. II-2}$$

where a is the well spacing. We now proceed to show that this function satisfies all of the boundary and source conditions. At each of the points $y = 0$, $x = 0, a, 2a, \ldots na$ there is a source of strength Q/T. To show this, consider that x and y are very small relative to the well spacing, a, so that the hyperbolic cosine and the cosine can be represented by the first two terms of their Taylor series expansions. Thus

$\cosh \frac{2\pi y}{a}$ equals approximately $1 + \frac{1}{2} \left(\frac{2\pi y}{a}\right)^2$

and $\cos \frac{2\pi x}{a}$ equals approximately

$1 - \frac{1}{2} \left(\frac{2\pi x}{a}\right)^2$. Making these substitutions

into equation 2 we find that in the neighborhood of a source

$$h = h_o - \frac{Q}{4\pi T} \log_e \frac{1}{2} \left(\frac{2\pi}{a}\right)^2 (x^2 + y^2)$$

$$= h_o - \frac{1}{2\pi} \frac{Q}{T} \log_e r - \frac{1}{4\pi} \frac{Q}{T} \log_e \frac{1}{2} \left(\frac{2\pi}{a}\right)^2 \qquad \text{Eq. II-3}$$

Differentiating Equation II-3 with respect to r, we obtain

$$\frac{dh}{dr} = - \frac{Q}{2\pi r T} \qquad \text{Eq. II-4}$$

which is the well known expression for the hydraulic gradient near a pumping well, except for the negative sign.

The slope of the piezometric surface in th y direction is:

$$\frac{\partial h}{\partial y} = - \frac{Q}{2aT} \frac{\sinh(2\pi y/a)}{\cos(2\pi y/a) - \cos(2\pi x/a)}$$

$$= - \frac{Q}{2aT} \frac{\tanh(2\pi y/a)}{1 - \frac{(\cos 2\pi x/a)}{(\cos 2\pi y/a)}} \qquad \text{Eq. II}$$

For large values of y, Tanh $\frac{2\pi y}{a}$ approaches and cosh $\frac{2\pi y}{a}$ increases rapidly, so that the above expression rapidly approaches as a lin

$$\frac{\partial h}{\partial y} = \frac{-}{+} \frac{Q}{2aT} \qquad \text{Eq. I}$$

which is independent of x.

This hydraulic gradient, for large values y, is the same as would be expected were the injected water coming from a line source coincident with the centerline of the wells and flowing equally in each direction.

Non-symmetrical potential distributions d to unequal sinks at $y = \pm \infty$ can be construct by superimposing a transverse linear flow, q per unit width, on the flow from the injection wells. The potential distribution is then

$$h = ho - \frac{Q}{4\pi T} \log_e \left(\cosh \frac{2\pi y}{a} - \frac{2\pi x}{a}\right) - \frac{q}{T} y \qquad \text{Eq. I}$$

Q and q are determined from values of the inland and seaward flow of fresh water from th line of wells. As an example of the use of Equation II-7, the difference between the wat level in one injection well and the piezometri surface midway between two such wells (out a line of wells) will be computed. Assume th 12 inch diameter wells are spaced at 500 fee Then at the point on the well circumference a on a line passing through all wells, $y = 0$, $x = .5$ ft., and

$$h_w = ho - \frac{Q}{4\pi T} \log_e \left[\cosh(0) - \cos \frac{2\pi .5}{500}\right.$$

$$= ho - \frac{Q}{4\pi T} \log_e \left[1 - \cos \frac{2\pi}{1000}\right]$$

This may be evaluated by noting that for small values of $\frac{2\pi x}{a}$ the cosine may be represented as:

$$\cos \frac{2\pi x}{a} \approx 1 - \frac{1}{2}\left(\frac{2\pi x}{a}\right)^2$$

and that $\log_e(x) = 2.303 \log_{10}(x)$. Thus

$$h_w = h_o - .863\frac{Q}{T} .$$

At a point midway between wells, $y = 0$ and $x = 250$ ft. Thus:

$$h_m = ho - \frac{Q}{4\pi T} \log_e \cosh 0 - \cos \pi$$

$$= ho - \frac{Q}{4\pi T} \log_e 2$$

$$= ho - .055 \frac{Q}{T}$$

The difference is:

$$h_w - h_m = (.863 - .055) \frac{Q}{T} = .808 \frac{Q}{T}$$

This value does not take into consideration losses from partial clogging of the wells.

It may now be seen that h_o corresponds to the height of the piezometric surface at a point one-fourth of the distance from one well to the next, where $\cos \frac{2\pi x}{a} = \cos \frac{\pi}{2} = 0$, and $\log_e \cosh 0 = \log_e 1 = 0$. (In general h_o is a datum elevation, and should be evaluated at a point where the actual water surface elevation is known.)

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APPENDIX C, PART II

AN ABSTRACT OF LITERATURE PERTAINING TO SEA-WATER INTRUSION AND ITS CONTROL

Sanitary Engineering Research Project
University of California, Richmond

Published as Technical Bulletin No. 1[illegible], I.E.R.
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PREFACE

PURPOSE OF STUDY

The intrusion of sea water into fresh water aquifers serving as major sources of water supply in coastal areas of California is a matter of great concern. The purpose of this study is to summarize the more important literature on the subject of sea water intrusion and methods for its control as a background for laboratory and field investigations of the phenomena involved. Such investigations are being conducted by the State Division of Water Resources through contracts with various agencies, including the University of California.

ORGANIZATION OF STUDY

The abstract of literature was made by Professor David K. Todd under the supervision of Professor T. Russell Simpson, faculty investigator on the Sea Water Intrusion study. This study was financed by the State Division of Water Resources and is a part of the activities of the Sanitary Engineering Research Projects of the University of California, of which Professor Harold B. Gotaas is Director.

In preparing the abstract all readily available and important papers bearing on the subject were summarized but no attempt was made to present a complete bibliography on sea water intrusion. Papers not directly applicable or which contain incidental information relative to sea water intrusion were omitted. In certain cases abstracts were taken from those in other papers, notably Water Supply Paper 537 (Brown, 1925). A few papers which were not available at the time are listed by author and title only.

Several of the abstracted papers deal with problems peculiar to the petroleum industry. There is, however, a similarity of problems in oil fields and in sea water intrusion zones. In oil fields gas cycling, water flooding, and salt water disposal all involve injection of fluids underground; hence, information valuable to sea water intrusion problems may be obtained from theoretical and practical solutions found in the oil industry. Similarly, studies of drilling muds for oil wells may be useful in evaluating properties of clay suspensions for cutoff walls.

The abstracts are divided into five parts by general subject matter and are listed alphabetically by author in each part. The material covered in each of the five parts is outlined below:

Part I - Reduction of Aquifer Permeability

Properties of bentonite suspensions (including drilling muds); grouting with asphalt emulsions, cement, chemicals, and plastics; colloidal properties; silt injection; and cutoff wall construction.

Part II - Sea Water Intrusion

Reports and analyses of sea water intrusion in coastal aquifers in California and throughout the world; Ghyben-Herzberg principle; cation exchange and diffusion of sea water; and criteria for recognition of sea water in ground water aquifers.

Part III - Injection and Recharge of Aquifers

Injection Wells - their behavior, construction, limitations, and maintenance; chemical composition

and treatment of recharge water; effect of recharge water on temperatures of aquifers; salt water injection in oil fields; theory of injection rates and volumes and well spacings; drainage wells; and fresh water barriers against sea water intrusion.

Part IV - Laboratory and Model Studies

Theory, construction, operation, and interpretation of capillary (two-dimensional), electrical, three-dimensional, gel, and color-tracer models for studying ground water flow; fluid flow analyses from tests of porous media and packings of uniform spheres; and laboratory tests on physical properties of waterbearing materials.

Part V - Ground Water Flow

Fundamentals of ground water flow; modifications of Darcy's Law; flow, compressibility, and pressure transmission in confined aquifers; two-fluid systems in porous media; and flow equations for drainage and artesian wells.

ACKNOWLEDGMENTS

Several individuals made valuable contributions to this report through advice or cooperation in locating material. Especially important contributions came from Professor H. A. Einstein, who furnished abstracts of the IUGG Assembly in Brussels; Professor H. B. Gotaas, who furnished several Dutch papers; Professor J. A. Putnam, who assisted in locating

pertinent papers in the petroleum field; Professor T. R. Simpson, who furnished papers on sea water intrusion in California and advised on the preparation of abstracts; and P. H. McGauhey, who assisted in preparing the material for publication.

PART I - REDUCTION OF AQUIFER PERMEABILITY

Alexander, J., Bentonite Colloid symposium monograph, vol. 2, pp. 99-105, 1925.

Presents a general discussion of bentonite, including definition, properties, occurrence and origin, composition, and practical uses.

Ambrose, H.A. and Loomis, A.G., Some colloidal properties of bentonite suspensions, Physics, vol. 1, no. 2, pp. 129-136, 1931.

A study of the swelling and gelling properties of bentonite dispersions has been made in connection with its use for drilling oil wells. It is shown that the emulsoid type of colloids present respond to changes in the pH of the dispersion medium with respect to rate of settling, viscosity and swelling of bentonites. Relaxation curves are given for bentonite suspensions and utilized to explain the suspension qualities of these soils for heavy mineral dispersions.

Anonymous, Foundation Stabilization, The Prepakt Reporter, 16 pp., November, 1949.

Describes general application of Intrusion mortar, composed of portland cement, Alfesil, Intrusion aid, sand, and water, to convert foundation gravels to Prepakt Concrete. Intrusion grout, containing the same materials except sand, is useful for stabilization of sands and finer materials. A brief discussion of methods and applications to particular jobs is included.

Anonymous, The Shellperm Process for Controlling the Flow of Underground Water, Shell Oil Company, New York, 8 pp., 1949.

Describes briefly the Shellperm process for controlling ground water flow. Shellperm is a patented asphaltic emulsion which is pumped through a pipe underground to the desired depth. The emulsion, emerging from the pipe, flows in all directions mixing with the sands and gravels. Chemicals in the emulsion cause the asphalt to separate out and coagulate in a sticky, plastic mass which clogs the pores. Principal equipment required included a two-inch steel pipe with a driving device, a specially designed driving point and nozzle, and a low pressure reciprocating pump. The process has been used in Egypt, Belgium, Holland, and England. In United States the first large scale test was a vertical underground barrier to prevent leakage through a diversion dam on the Santa Ana River (see Blakeley and Endersby, 1948). This cut-off wall resulted in a saving of about 0.88 acre-feet of water per day. The cost of the process is a function of the quantity of emulsion required, the cost of emulsion, chemicals, and transportation, local labor costs, and depth of barrier constructed.

Anonymous, Use of silicates of soda, Silicate P's and Q's, Philadelphia Quartz Co. of California, Berkeley, vol. 30, no. 10, 1950.

The use of sodium silicate and calcium chloride solutions to seal leaks in an underpass in La Grande, Oregon, is described. Chemical solidification was achieved for about one foot below the concrete base by pumping the solutions at 400 psi pressure through $1\frac{1}{2}$ inch holes into the sand and gravel substructure. Freezing during the repair work reopened some of the leaks, but the following winter failed to cause any further leakage.

Anonymous, Digging a trench to keep water out under Columbia River levees, Western Construction, vol. 27, no. 6, pp. 107-108, June, 1952.

The construction of 6.3 miles of sub-levee cutoff trenches ranging in depth to 60 feet along the Columbia River are described. The contractor was given a choice of three options for constructing the cutoffs:

(a) Minimum trench width of 6 feet with a moderately well-graded (3 in. to silt) backfill to be pushed into the trench in a semi-fluid state to prevent segregation.

(b) Minimum trench width of 3 feet and using a more closely graded core material than in (a).

(c) Minimum trench width of 3 feet backfilled with material specified in (a) plus a bentonite slurry.

The first option was selected by the contractor because tests had demonstrated that the gravels encountered would stand nearly vertical for short lengths of time and because a 6-foot trench width gave greater scope in selection of equipment. The excavated material was mixed with borrow material along the trench by bulldozers and motor scrapers, then wetted to a consistency of wet concrete, and finally bulldozed back into the cutoff trench.

Blakeley, L.E. and Endersby, V.A., Prevention of underground leakage, Jour. Amer. Water Works Assoc., vol. 40, no. 8, pp. 873-882, 1948.

Describes application of Shellperm process (see Anonymous, 1949) for impermeabilizing aquifers along the Santa Ana River, California. The injected asphaltic emulsion was placed along a 350 foot length in depths varying from 5 to 30 feet. The first line of injection holes were spaced four feet apart and emulsion was pumped in at a rate of 10 gallons per vertical foot. The second line of holes, offset about 1.5 feet and staggered in relation to the first line holes, had emulsion pumped in at a rate of 20 gallons per vertical foot. Average pressures at the head of the injection pipe were 20-30 psi. The entire operation, requiring 1.75 months, closed off an area of 4894 sq. ft. and used 32,400 gallons of emulsion. Underflow tests using dyes indicated the cut-off wall provided a water saving of more than 0.66 acre-feet per day.

Bond, D.C., Positive colloid muds for drilling through heaving shale, Petroleum Technology, vol. 7, no. 1, 10 pp., 1944.

Describes positive colloid mixtures and their usefulness as drilling muds. The stability, specific gravity, viscosity, gel strength, filtration, and effects of high temperatures on these mixtures are discussed. A bibliography of 49 items is appended to the paper.

Cary, A.S., Walter, B.H., and Harstad, H.T., Permeability of Mud Mountain Dam core material, Trans. Amer. Soc. of Civil Engineers, vol. 108, pp. 719-737, 1943.

Reports the results of an investigation inauguarated to determine the permeability-void ratio relationships of the borrow-pit material as a basis for the distribution of material in Mud Mountain Dam. It was found that slight changes in moisture content had pronounced effects on the permeability of the material. The testing apparatus is illustrated and the results presented in graphical form.

Cole, D.W., Stabilizing constructed masonry dams by means of cement injections, Trans. Amer. Soc. of Civil Engineers, vol. 101, pp. 714-766, 1936.

Methods, results, and costs are presented on cement grouting of three dams in India to reduce seepage losses.

Corps of Engineers, U.S. Army, Efficacy of bentonite for control of seepage, The Experiment Station Bulletin (Soil Mechanics), U.S. Waterways Experiment Station, Vicksburg, Miss., vol. 2, no. 1, pp. 2-5 1938.

Four test phases are reported upon:

(a) Experiments to determine a suitable method of mixing bentonite with water to form a grout;
(b) Tests conducted in a pyralin cylinder to determine optimum concentration of grout, for impregnating the sand used in the tests;
(c) Extension of (b) using optimum concentration developed therein for grouting bed of sand contained in a rectangular box; and
(d) Tests conducted on model levee, using bentonite as grout, blanket, and cut-off wall.

The results of the four tests are briefly described together with comments and conclusions.

Corps of Engineers, U.S. Army, Bentonite grout tests, The Experiment Station Bulletin (Soil Mechanics), U.S. Waterways Experiment Station, Vicksburg, Miss., vol. 3, no. 2, pp. 6-7, 1939.

Experimental data of stability of slag treated with bentonite grout are presented. Test results showed that a 12.5% bentonite grout was stable for hydraulic gradients less than 4.0. Recommends that this grout not be used for cut-offs in the slag formation for flood wall purposes.

Corps of Engineers, U.S. Army, Soil-bentonite mixtures, The Experiment Station Bulletin (Soil Mechanics), U.S. Waterways Experiment Station, Vicksburg, Miss., vol. 4, no. 1, pp. 8-9, 1940.

Materials, apparatus, tests, and results are briefly presented of permeability tests on soil-bentonite mixtures. Concluded that the bentonite used in soil-bentonite mixtures should be of a grain size equal to or less than the treated soil and that the mixtures should be confined when possible.

Corps of Engineers, U.S. Army, Conference on Control of Underseepage, U.S. Waterways Experiment Station, Vicksburg, Miss., 148 pp., 1945.

Contains five papers on conditions conducive to detrimental underseepage; four papers on examples of detrimental underseepage and description of remedies; six papers on methods for control of underseepage, including impervious blankets, longitudinal drains, drainage blankets, berms, sublevees, and relief wells; and five papers on foundation exploration, including electrical resistivity and permeability tests.

Corps of Engineers, U.S. Army, Report on Experimental Bituminous Grouting of Pervious Foundation, Soils, Foundation and Frost Effects Laboratory, New England Div., Boston, 27 pp., 1950.

Describes an experimental field investigation of the grouting of pervious foundation soils with asphalt emulsion for the purpose of reducing foundation permeability conducted at Mansfield Hollow, Connecticut. Grouting was performed below ground water in stratified sands and gravels to a depth of 35 feet, through 10 injection holes spaced equi-distantly along the circumference of a 10-foot diameter circle and through one hole at the center of circle. Pumping tests were performed before and after grouting to determine reduction in seepage due to grouting. After injection, the foundation was explored to below the zone of grouting by means of five drive sample borings spaced along the periphery of the injection circle and by open trench excavation which was carried to approximately 4 feet below ground water. The results of the investigation indicate that seepage was markedly reduced where the asphalt emulsion penetrated the soil strata, but that the method of injection used was not suited to securing uniform distribution in stratified formations of such variable permeability as encountered in this project.

Craft, B.C. and Moncrief, C.M., Effect of arsenates on the viscosity of drilling muds, Petroleum Technology, vol. 8, no. 6, 3 pp., 1945.

Present results of tests on bentonite clay showing that treatment with tetrasodium pyroarsenate caused the same reduction in viscosity as did complex phosphates.

Davis, C.W., The swelling of bentonite and its control, Industrial and Engineering Chemistry, vol. 19, no. 12, pp. 1350-1352, 1927.

The effect of different liquids on the swelling of bentonites shows that lubricating oil, kerosene, and gasoline prevent swelling and leave a hard, granular residue and that, while dilute solutions of various salt solutions retard swelling, saturated solutions of specific salts are required to obtain the maximum effect for each type of bentonite. Change in temperature from 1° C to 94°C accelerates the rate of swelling but has little effect on the final volume for the solutions used. Increase of acidity or alkalinity depresses the swelling of bentonite. Equivalent quantities of neutral salts reduce the swelling of bentonite to about the same degree but univalent anions are slightly more effective than polyvalent anions. The specific dehydration effect of several cations at certain given normalities, although small, follows the same order for chlorides and sulfates.

Davis, C.W. and Vacher, H.C., Bentonite - its Properties, Mining, Preparation, and Utilization, Technical Paper 438, U.S. Bureau of Mines, Washington, D.C., 51 pp., 1928.

The principal deposits of bentonite are described together with the mining and production procedures involved. The properties of bentonite are investigated, including its general characteristics, composition, effects of liquids on swelling and disintegration, action in water, control of swelling, and loss of colloidal properties on heating. The effect of sodium chloride and calcium chloride solutions are discussed briefly. The identification and classification of commercial bentonites and the application of the properties of bentonites for commercial purposes are explained. The paper concludes with an extensive discussion of the present and proposed uses for bentonite.

Davis, R.E., Jansen, E.C., and Neelands, W.T., Grouted gravel fill and precast slabs provide new face for Barker Dam, Civil Engineering, February, 1948.

Describes new facing for Barker Dam composed of a layer of pre-cast concrete slabs bonded to the old face by aggregate and cement grout. The aggregate used contained sand from 0 to No. 16 and gravels from 5/8 to 4½ inches. Pipes were inserted in the aggregate between the old face and the new slabs and hoses were connected to the pipes for pumping the grout into the voids. The grout was proportioned to contain 2 sacks of cement, 100 lbs. of Alfesil, 3 lbs. of Intrusion

Agent, 300 lbs. of sand, and about 16 gals. of water. The average temperature of the grout was purposely kept low and left the mixer at 57°F.

Endersby, V.A., Construction Report on Shellperm Project at Diversion Dam of the Santa Ana Valley Irrigation Company, Report No. S-13023-R, Shell Development Company, Emeryville, California, 15 pp., 1948.

Describes and illustrates the equipment employed and the operations executed in a Shellperm injection project along the Santa Ana River to limit ground water losses (see Blakeley and Endersby, 1948). It is noted that anomalies in distribution of the emulsion near the surface produced deficient areas in the barrier which account for residual ground water flow. Where the barrier was complete, the shut-off was satisfactory.

Endersby, V.A., The Shellperm Process - Witherby Street Undercrossing, San Diego, California, Report No. S-13035-R, Shell Development Company, Emeryville, California, 4 pp., 1948.

Describes use of Shellperm to seal leaks in a street undercrossing in San Diego. Three main flow leaks, caused by ground water seepage, were treated and it is estimated that 70-80% of the total leakage in the structure was shut off. Observation $2\frac{1}{2}$ months after treatment indicated that the treatment was holding and that no noticeable plastic flow was taking place. It was necessary to use an asphalt concentration of 60% rather than the normal 30% and break it quickly in the structure.

Endersby, V.A., Preliminary Report on Proposed Shellperm Project at Attleboro, Mass., Report No. S-13065-R, Shell Development Co., Emeryville, California, 14 pp., 1949.

Describes cooperative field trials of the Shellperm asphaltic emulsion process for sealing aquifers near Attleboro, Mass. to determine its feasibility, probable cost, and effectiveness. Four injections were made at the site and it was found that the formation will take injection at a satisfactory average rate and that the results satisfactorily proved the process. Details of the topography and hydraulics of the location, the extent of water loss, nature of the soils, organization and equipment for the test, injection tests, and permeability tests of the treated soil are included in the report. Photographs of the test operation are presented.

Endersby, V.A., Shellperm Injection Equipment, Report No. S-13096, Shell Development Co., Emeryville, California, 5 pp., 1949.

Describes the general nature of the Shellperm process, preparation of the emulsion, pumping of the emulsion, the injection point, the components of a complete sample injection unit, possible improvements in the injection unit, and chemical and engineering controls of the operation. Figures illustrate the Shellperm injection point and pumping unit.

Fancher, G. and Whiting, R.L., Response of a Gulf Coast drilling mud to chemicals, temperature, and heat-treatment, Petroleum Technology, vol. 6, no. 2, 15 pp., 1943.

Describes the effects of water dilution, treatment with complex polyphosphates, temperature, and time of heating upon the rheological properties of a specified colloidal drilling mud. It was found that water plays an important part in chemical treatment and that sodium acid pyrophosphate and sodium tripolyphosphate were more efficient than other complex polyphosphates for chemical treatment. Muds treated with either of these chemicals manifested maximum reduction in viscosity and minimum filtration rates at low concentrations. Furthermore, mud treated to minimum viscosity with either of these two chemicals was virtually unaffected by heat-treatment.

Farris, R.F., Effects of temperature and pressure on rheological properties of cement slurries, Petroleum Technology, vol. 3, no. 2, 14 pp., 1940.

Investigates the behavior of cement slurries under high temperatures and pressures for their use in deep-well cementing operations. The apparatus and test results are described in detail. Concludes that high temperatures and pressures accelerate the stiffening and setting of a cement slurry.

Gray, G.R., Foster, J.L., and Chapman, T.S., Control of filtration characteristics of salt-water muds, Petroleum Technology, vol. 4, no. 5, 9 pp., 1941.

Describes experimental tests on salt-water drilling muds, which show that the desired characteristics of the muds can be markedly improved by the addition of natural gums, seaweeds, or gelantinized starch. Test results, presented graphically, show the changes in viscosity and filtration which occur when the various substances are added to drilling muds.

Hefley, D.G. and Cardwell, P.H., Use of plastics in water control, The Petroleum Engineer, vol. 15, no. 3, pp. 51-54, 1943.

Describes use of plastic plugging agents for making aquifers impermeable to water in oil fields. Several chemical solutions have

been suggested, all of them clear liquids containing no suspended matter and which undergo polymerization, condensation, or association reactions until the whole liquid becomes an insoluble, strong, solid plastic when treated or subjected to temperatures encountered in oil wells. The temperature of hardening can be adjusted by changes in the chemical composition of the plastic. Examples of suitable liquid plastics are unpolymerized styrene, vinylidene chloride, partially condensed phenol-formaldehyde, vinyl esters, and ester of maleic acid with diethylene glycol. Data from 100 plastic plug jobs in Texas show 65% gave complete water shut off, 18% gave 50-100% shut off, and 17% gave less than 50% shut off. Details of the well-plugging treatment are described and illustrated.

Hogentogler, C.A. and Willis, E.A., Essential considerations in the stabilization of soil, <u>Trans. Amer. Soc. of Civil Engineers</u>, vol. 103, pp. 1163-1192, 1938.

The underlying principles involved in soil stabilization and the possible means of its accomplishment are discussed. Particularly stressed is the application of the colloidal phenomena of adsorption and base exchange as they affect:

(a) Particles of soil, sand, crushed rock, gravel and slag coated with films of air, water, soluble chemicals, and binders not soluble in water;
(b) The relative adhesion between solids and films; and
(c) The effect of the chemical composition of aggregate and binders and the ions on the surfaces of the solid particles.

Included as an appendix is a bibliography of 36 items concerned with the various treatments and required tests, including: graded soil mixtures, calcium chloride, sodium chloride, asphalt emulsion, asphalt mixing method, tar, portland cement, sulfite liquor, molasses, calcium silicate, electro-chemical treatment, application of heat, mechanical analysis, routine soil tests, and Proctor tests.

Jenny, H., <u>Properties of Colloids</u>, Stanford University Press, 136 pp., 1938.

Contains an excellent summary in outline form of colloids and their properties. Major topics covered include, boundary phenomena on liquids, adsorption of gases on solids, adsorption of liquids by solids, adsorption of electrolytes and ionic exchange, types of forces and chemical bonds in adsorption phenomena, electrical properties of colloids, flocculation and its relationship to potential and exchange adsorption, Brownian movement, colloid optics, viscosity, gels, and swelling.

Jenny, H. and Reitemeir R.F., Ionic exchange in relation to the stability of colloidal systems, Journal of Physical Chemistry, vol. 39 no. 5, pp. 593-604, 1935.

Presents experimental results of studies relating ionic exchange, zeta potential, and flocculation value for colloidal clay systems. The experimental techniques involved in the measurement of the three phenomena are described. The principle relations investigated included zeta potentials as influenced by charge and size of the adsorbed cations, zeta potential and ionic exchange, zeta potentials in relation to flocculation values, flocculation values as affected by charge and size of the flocculating ions, flocculation values and exchange adsorption, flocculation by X-rays, and mechanism of flocculation with special reference to the role played by ionic exchange. Concludes that colloidal particles with high potentials carry adsorbed ions which are easily exchangeable, that the flocculation value of a given electrolyte increases potentially with the zeta potential, that for monovalent cations the flocculation value is inversely proportional to the ion size, and that the flocculating power of a cation is a positive exponential function of its ionic exchange ability.

Johnson, G.E., Silt injection checks seepage losses from water-supply canal in Nebraska, Engineering News-Record, vol. 148, no. 6, pp. 32-34, 37, 1952.

Describes use of injection wells to stop seepage from unlined canals in wind-deposited loess soils. A slurry of graded loess was injected into wells spaced 50 feet apart. The purpose of the injection was to consolidate the porous loess soil and reduce the moisture content of the saturated soil (from seepage) from 30% to 12-14%. Because it was found that seepage could be checked up to 80 feet from a well, no wells were sunk in the canal but only along each side of the canal. The slurry was injected under pressures of 80-125 psi increasing occasionally up to 270 psi. Quantities injected varied from 15 to 300 cu. yd. per well. Wells consisted of 6-inch holes 150 feet in depth with a 2-inch pipe 40 feet long inserted in the top and surrounded by gravel to receive the slurry. Costs equalled about $1500 per day or about $1.00 per cu. yd. of injected slurry. Details of the construction operation, including equipment employed, are described and illustrated.

Kennedy, H.T., The control of gas-oil and water-oil ratios by chemical treatment, Drilling and Production Practice, American Petroleum Institute pp. 214-220, 1941.

Discusses briefly the practical applications that have been made of chemical treatments in sealing off water from drilling wells and from producing wells. States that treatments up to four years old have proven satisfactory.

Lambe, T.W., Stabilization of soils with calcium acrylate, Journal of the Boston Society of Civil Engineers, vol. 38, no. 2, pp. 127-154, 1951.

Summarizes an extensive search for a chemical which can solidify soils. The best solidifier developed is calcium acrylate. The treatment consists of adding calcium acrylate monomer to soil, then polymerizing the acrylate with a redox catalyst to form a flexible product with significant tensile strength which can withstand the effect of water. The introduction contains an excellent summary of various chemical methods of treating soils, including chlorides, portland cement, bitumen, resins, sodium silicate, clay, and others. The calcium acrylate stabilization method is presented in considerable detail. The theory of treatment, stabilization procedure, and further improvements of the process are explained. Extensive tests of the engineering properties of the treated soils were conducted covering volume changes, strength and flexibility tests, and effects of water and soil composition. The treatment has been successful on all soils tested and the product formed has enough tensile strength to withstand water penetration.

Lee, C.H., Sealing the lagoon lining at Treasure Island with salt, Trans. Amer. Soc. of Civil Engineers, vol. 106, pp. 577-607, 1941.

The construction of a water-tight lagoon using a clay layer is described. Upon completion it was found that infiltration losses exceeded 1 inch per day. However, after pumping sea water into the lagoon and out again, the fresh water loss was about 0.10 inch per day. Extensive laboratory tests of the colloidal processes affecting the permeability of the clay blanket were conducted by G.B. Bodman at the University of California and this work is summarized in a discussion of the paper.

Lehnhard, P.J. and Reimers, H.A., Chemical formation plugging, The Oil Weekly, vol. 94, no. 5, pp. 15-20, 1939.

Presents field results of chemicals used to plug water-bearing formations in oil wells. Discusses the many types of chemicals which may be used, their properties, and relative merits. Some of the chemicals mentioned include: sodium silicate, sodium carbonate, and sulphuric acid (all of which form precipitates when in contact with brines); antimony trichloride, silicon tetrachloride, superfatted soaps, and finally divided cements in nonaqueous suspension; solidifying naphthalene vapor; and self-hardening liquid chemical mixtures. Describes the mechanics of setting of the last-named mixtures. The application of formation plugging to control water coming in oil wells is mentioned and illustrated. The mechanics of plugging wells is briefly described.

Lewin, J.D., Grouting with chemicals, Engineering News-Record, vol. 123, no. 7, pp. 221-222, 1939.

Describes process of soil consolidation by grouting with a combination of sodium silicate and calcium chloride. Discusses penetrating

action of the chemicals, how the chemicals act, method of injection, and gives examples of application - stopping seepage, improving foundations, and preventing movement of quicksand.

Loomis, A.G., Ford, T.F., and Fidiam J.F., Colloid chemistry of clay drilling fluids, Petroleum Technology, vol. 3, no. 2, 12 pp., 1940.

Presents a comprehensive analysis of the effects of various chemicals on the rheological properties of drilling muds. It is shown that viscosity-reducing chemicals are adsorbed on specific surfaces of the clay particles and that they reduce the viscosity of suspensions by destroying the portion of the absolute gel strength that is attributable to adhesion between particles. On the other hand salts are not adsorbed but cause gelation and coagulation because of induced changes in the interparticle ionic atmospheres. Results of a comparison of the viscosity-reducing effects of the polyphosphates with other standard viscosity reducers show that sodium pyrophosphate is one of the most effective. However, it is found that among the polyphosphates the pyrophosphate, which has the lowest molecular weight, produces the least permanent effect and that as the molecular weight increases to the decaphosphate there is a corresponding increase in the permanence of the viscosity lowering. This behavior suggests diffusion into inner particle surfaces, the rate of which decreases as the size of the adsorbed molecules is increased. Proof is given that the tannate and pyrophosphate ions are adsorbed on the same surfaces of the clay particles and that the polyphosphates produce equal viscosity-lowering effects per molecule. The application of thrixotropy and gel strength to drilling problems is discussed in detail. From X-ray analysis of clay structures, chemical formulas are written for colloidal clay particles and used to explain most of the phenomena shown by clay suspensions. Finally, it is shown that the assumption of edge to edge contact to form structures in clay colloidal systems is in accordance with the experimental facts.

Machis, A., Experimental observations on grouting sands and gravels Trans. Amer. Soc. of Civil Engineers, vol. 113 pp. 181-212, 1948.

Presents results of laboratory research on cement slurry for use in grouting well casings to prevent contamination of fresh water by encroaching sea water. Detailed laboratory studies showed that for sands having a coefficient of permeability less than 3000 gal. per day per sq. ft., the pores of the sand will be too small to allow the cement grains to enter; hence a filter cake is formed on the sand. For sands having a permeability over 3000 the cement will penetrate the formation; the amount of penetration depending upon the ratio of the pore size to the cement particle size, upon the slurry concentration and upon the manner in which the pressure is applied to the slurry application.

Montgomery, P., A survey of the cementing jobs to shut-off salt water in East Texas Field, <u>The Oil Weekly</u>, vol. 92, no. 1, pp. 34-42, 1938.

Describes in detail the 12 methods in use for cementing oil wells to shut-off entrance of salt water. Mentions most of the jobs are using slow setting gel-forming cement (containing admixtures of bentonite) because they do not flow as readily as ordinary cement and hence give a better plugging action under pressure.

Polivka, M., Soil Solidification by means of chemical injection, M.S. Thesis, University of California, Berkeley, 33 pp., 1948.

The methods of soil solidification by means of chemical injection are described. The limitations of the methods, the comparison of various methods by laboratory tests, and the recommended procedures for finding optimum proportions of materials and injecting the chemicals are presented. From its development in Europe to its present-day use in this country, the history of soil solidification is given. The chemicals involved in the process, primarily sodium silicate and calcium chloride, are discussed, followed by descriptions and proportions required for the one-solution and two-solution methods. All laboratory tests were performed on sands with sizes ranging from those passing a No. 48 sieve to those retained on a No. 14 sieve. Two types of silty loam were tested, but the required pressures and times of penetration for injection were impractical. The laboratory equipment and test procedures employed are outlined. Results of numerous laboratory tests are presented, including:

(a) Proportions for maximum economy of materials;
(b) Effect of amount of silicate and curing conditions upon physical properties of solidified soil;
(c) Effect of various grades of silicates upon physical properties of solidified soils
(d) Effect of dilution of silicate upon physical properties of solidified soil;
(e) Effect of concentration of calcium chloride upon physical properties of solidified soil;
(f) Effect of injection pressures upon time and depth of penetration,
(g) Limitations of the two-solution method,
(h) Exploratory tests for single solution copper sulphate solutions; and
(i) Exploratory tests for single solution sodium bicarbonate solutions.

From the test results, concludes that due to the low viscosities of the silicates, they can be injected into materials where other materials such as cement, bitumen, or bentonite might fail to penetrate. The lower limit of the maximum grain size for the two-solution method is about a sieve No. 48 and for the one-solution method is about a No. 100 sieve. The two-solution method gave greater strengths, but the one-solution method appears to be the most practical. The selection of the method to be used would depend upon the field conditions and preliminary laboratory

testing is recommended. An extensive bibliography is appended on solidification by chemical injection containing 80 items, the majority of which are French, German, or Russian papers.

Polivka, M., Running sand chemically solidified, Western Construction News, July 15, 1949.

Chemical solidification by the "one solution" and "two solution" methods is described. The problems of the correct solutions, equipment, and premature sealing are mentioned. The "one solution" method is illustrated in constructing lighting towers for a stadium in loose sand. Diagrams and photographs show the pipe injection pattern and the sand solidification zone for the foundation supports.

Power, H.H. and Houssiere, C.R., Chemical treatment of bentonitic suspensions and the relationship to the heaving-shale problem. Petroleum Technology, vol. 4, no. 6, 16 pp., 1941.

Includes a detailed study of the effect of chemicals in bentonitic suspensions and the most plausible theoretical explanations for the behavior of the clay minerals. Experimental results cover effects of various ions, viscosity, filtration, pH, permeability, and particle size.

Rhodes, A.D., Puddled-clay cutoff walls stop sea-water intrusion, Civil Engineering, vol. 21, no. 2, pp. 21-23, 1951.

Describes construction equipment and methods of placing puddled clay cutoff walls around oil field near Long Beach, California. Procedure consisted of digging a 32-in. wide trench to depth of underlying impervious strata by means of a ladder-type trenching machine using closed clamshell buckets of alloy plate steel. The trench walls were maintained by keeping the trench filled with slurry consisting of water, clay, and bentonite. The slurry was kept in condition by circulation and by passing it through a desander. The trench was backfilled with clay using a specially designed puddling ram which forced the clay to the bottom of the trench and displaced the slurry. Numerous core samples and test plots showed the cutoff wall to be solid, uniform, and water-tight. Indicates that equipment used for this job could not go deeper than 45 ft., but that much greater depths could be dug and placed economically where the volume of work was sufficient to justify the expense of developing necessary equipment.

Riedel, C.M., Chemicals stop cofferdam leaks, Civil Engineering, vol. 21, no. 4, pp. 23-24, 1951.

Describes chemical solidification process employed to develop solid foundation for a bridge pier near Kuttawa, Kentucky. Cofferdam for pier foundation encountered loose, soft rock, sand boils, and artesian

water. Chemicals used were sodium silicate (water glass) and calcium chloride, which combine to form a silicic gel. This gel forms a soft sandstone which is impervious to water and has bearing strengths up to 50 tons per sq. ft. The process is not applicable to strata containing more than 25 per cent of clay, silt, or sand passing a 125-mesh sieve. The chemicals were pumped in alternately through pipes until the water and sand boils were sealed off successfully.

Robeson, F.A. and Webb, W.E., Cofferdam grouting at Jim Woodruff Dam, Engineering News-Record, vol. 145, no. 1, pp. 35-37, 1950.

Describes pre-excavation grouting to reduce leakage during subsequent cofferdams construction. Several trial grout mixes were tried until a satisfactory one was found, containing a 50-50 mixture of two sands to obtain a well-graded mix together with Alfesil and Intrusion Aid, which are patented products of the Prepakt Concrete Co., Cleveland, Ohio. Alfesil is a pozzolanic fly-ash which acts as a filler, prevents the flocculation of cement particles, and combines with lime released by the cement to form a stable compound. Intrusion Aid creates a film of cement particles that gives the mix the properties of a colloidal suspension, eliminates setting shrinkage, produces an expansion of the grout just prior to initial set, and prevents early stiffening of the mortar. The grout procedures, plant, and results are discussed.

Ross, C.S. and Shannon, E.V., The minerals of bentonite and related clays and their physical properties, Journal of the American Ceramic Society, vol. 9, no. 2, pp. 77-96, 1926.

Describes the occurrence, structure, and chemical composition of bentonites. States that the high adsorptive power of bentonites comes from the physical form of the particles. Thus the micaceous structure and easy cleavage, which give very great surface area and the felt-like texture, facilitate permeability. Closes with a short discussion of beidellite and of the optical properties of various clays.

Simonds, A.W., Final foundation treatment at Hoover Dam, Proc. Amer. Soc. of Civil Engineers, Sep. No. 109, 22 pp., 1951.

To prevent seepage and reduce uplift pressures, an extensive grouting program of foundations of Hoover Dam was undertaken. A combination of stage and packer grouting proved most effective. For packer grouting, drill holes of $1\frac{1}{2}$ - 2 inches in diameter, 5 to 20 feet apart, were employed. The procedure required the use of extremely thin grout mixtures and pressures as high as could be safely maintained without displacing the foundation rock. The water cement ratio of the grout mixture ranged from 15:1 to 5:1, and pressures of 550-600 lb. per sq. in. were used for pumping. In areas of hot alkaline water a commercial retarder for the grout mixtures proved advantageous.

Simonds, A.W., Lippold, F.H., and Keim, R.E., Treatment of foundations for large dams by grouting methods, Trans. Amer. Soc. of Civil Engineers, vol. 116, pp. 548-574, 1951.

Presents fundamentals of grouting as applied to foundations of large dams. Discusses and illustrates grouting equipment, systems, and grouting procedures. Cement, chemical, and asphalt grouts are described together with their applicability. Special admixtures and their purposes are mentioned for use in particular cases. The four general methods used in foundation grouting - single-stage, grouting by packers, successive-stage, and multiple grouting - are defined.

Smith, W.H., et al., Lateral earth pressures on flexible retaining walls - a symposium, Trans. Amer. Soc. of Civil Engineers, vol. 114, pp. 409-538, 1949.

Model tests of earth pressure on flexible bulkheads are described, including tests with submerged sand backfills, with clay and with sand-clay mixture backfills placed in fluid condition, and with sand dikes and with vertical sand blankets of varying thickness backfilled with fluid clay. Results of laboratory soil classification tests and of shear strength determinations are also given. The results are applied to quay wall designs.

Sommer, H.J. and Griffin, R.L., Shellperm process - behavior of Shellperm injections in a sand with varying water contents, Report No. S-13050. Shell Development Company, Emeryville, California, 7 pp., 1948.

Describes a series of experiments performed to study the distribution of Shellperm in saturated, damp, and dry sand. It was found that an emulsion containing 30% by weight asphalt could be successfully injected in all sand tested with resulting firm set, little loss of emulsion by run down, and no break out to the surface, if the following conditions are met: a 1% casein stabilizer is added to the emulsion; minimal pressures are used to avoid channelling and break outs; and the ethyl formate, added to the emulsion to produce hardening, should be first mixed with the diluting water to avoid lumps forming in the mixture.

Warren, J.A., Effect of salt water on bentonite, Mining and Metallurgy, vol. 7, no. 236, p. 349, 1926.

Describes an experiment which shows that bentonite does not adsorb saturated salt solutions and no apparent volume changes occur. This is quite different than the reaction with pure water, where all the water is adsorbed and the clay swells to several times its original volume. The results of this experiment were applied in well drilling. By dumping 20[illegible] lbs. of salt in a well which encountered a bentonite layer, no caving of the well occurred.

Weber, A.H., Correction of reservoir leakage at Great Falls Dam, <u>Trans. Amer. Soc. of Civil Engineers</u>, vol. 116, pp. 31-48, 1951.

Describes grouting by injecting hot asphalt and cement into 608 holes to intercept water-bearing rock fissures. Leakage was reduced 98% by this treatment. Grout holes were drilled with diamond bits with an average cost of $5.30 per linear foot. Oxidized petroleum asphalt was used for grouting. Heaters of 500-gal. capacity and air-driven, double-acting reciprocating pumps were used for melting and placing the asphalt. All packer settings were made as close as possible to the water-bearing cavities to obtain a maximum seal. Pumping rates varied from 40 to 80 cu. ft. per hr. A diagram showing the complete asphalt grouting equipment setup is presented. Asphalt temperatures varied from 300 to 350°F. Total cost for asphalt and placement amounted to $1.35 per cubic ft. Cement grouting by packer setting was also employed with pressures similar to those used with the asphalt. Calcium chloride was often added to accelerate the set of cement with water - cement ratios of 0.6. Total cost of cement and placement was $1.19 per bag.

Wherry, E.T., Bentonite as one-dimensional colloid, <u>The American Mineralogist</u>, vol. 10, no. 5, pp. 120-123, 1925.

Suggests that colloidal particles of bentonite are plates of microscopic breadth but of colloidal thickness. This concept may be used to explain the fact that bentonite swells to many times its dry volume in the presence of water, because the broad surfaces are capable of exerting greater capillary attraction than ordinary colloid grains. Similarly, the plasticity of bentonite may be connected with sliding of these plates over one another and lubricated by the water films, while its high adsorptive powers may be attributed to the large surface area of the particles relative to the particle mass.

d'Andrimont, R., Notes sur l'hydrologie du littoral belge (Notes on the hydrology of the Belgian coast). Soc. Geol. Belgique Annales, vol. 29, pp. M129-M144, Liege, 1902.

Describes the geology of the Belgian coast and refers to Herzberg's theory and modifications in its application to the topography and geology in that region. This includes the problem of an unconfined aquifer overlying a confined aquifer and the relation of sea water intrusion into both aquifers. A figure from this paper is shown on p. 36 of the paper by Brown 1925 (which see) illustrating the salt water intrusion into the two aquifers (Abstract from Brown, 1925).

d'Andrimont, R., Contribution a l'etude de l'hydrologie du littoral belge (A contribution to the study of the hydrology of the Belgian coast). Annales de la Societe Geologique de Belgique, vol. 30, pp. M3-M43, 1903.

Discusses the geologic and hydrologic aspects of the coastal dune area with respect to the availability, movement, and quality of the water found therein. Concludes that there is a tremendous quantity of water available in the dune area, that most of it flows through the dune aquifers toward the sea, that it flows rapidly enough to prevent sea water infiltration, that a large volume of this water may be used without causing sea water intrusion, and that the water found in the dunes is potable.

d'Andrimont, R., Note complementaire a l'etude hydrologique du littoral belge (Supplementary note on the hydrology of the Belgian coast). Annales de la Societe Geologique de Belgique, vol. 31, pp. M167-M183, 1904.

Discusses the paper by Van Ertborn, 1902 (which see), and describes the Herzberg theory as applied to fresh-salt water relationships in coastal areas. Also discusses the work of Dubois, 1905 (which see), at the end of the paper.

d'Andrimont, R., L'allure des nappes aquiferes contenues dans des terrains permeables en petit, au voisinage de la mer (The nature of ground water contained in freely pervious aquifers adjacent to the sea). Annales de la Societe Geologique de Belgique, vol. 32, pp. M101-M114, 1905.

Discusses the results of research work done in Holland by Pennink, 1904 (which see), on the fresh-salt water relationships in coastal dune areas and shows how the same results apply to the author's previous investigations of this problem on the Belgian coast.

Anonymous. Le probleme de l'infiltration sonterraine dans le polders de la Hollande (The problem of underground infiltration in the polders of Holland) Commission des Eaux Sonterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951, (Unpublished).

The level of the Holland polders descends sometimes several meters to below sea level or that of the surrounding water. With the presence of large areas of permeable sub-soil beds, the water can enter by infiltration, and must then be removed by pumping. Sometimes the volume of water removed originating from precipitation is small compared to that from infiltration. It is thus important to forecast the infiltration into new polders. A discussion is given of the factors to consider and the methods of forecasting.

Badon Ghyben, W., Nota in verband met de voorgenomen put boring nabij Amsterdam (Notes on the probable results of the proposed well drilling near Amsterdam). K. Inst. Ing. Tijdschr., 1888-1889, p. 21, The Hague.

Contains first enunciation of the principle, later discovered independently by Herzberg, that fresh ground water floats above salt water because of its lower density. (Abstract from Brown, 1925).

Banks, H.O. and Bookman, M., Proposed investigational work for control and prevention of sea-water intrusion into ground water basins. Report to State Water Resources Board, Div. of Water Resources, State of California, 33 pp., Sacramento, 1951.

Summarizes status of sea water intrusion in California, possible methods of control of sea water intrusion, and prior and current investigational work, including: recharge through injection wells by Los Angeles County Flood Control District, University of California, and other agencies; surface spreading; parameters of intruding saline waters; and subsurface barriers by grouting and by puddled clay cutoff walls. States that only the methods of pressure-ridges above sea level and construction of subsurface barriers appear to be worthy of consideration at this time. On this basis the factors to be determined by an investigational program are mentioned including: factors concerned with the nature and occurrence of sea water intrusion, pressure ridges, injection wells, use of reclaimed waters, and subsurface barriers. Also outlines factors to be considered in selection of areas for field investigation: adequate geologic and hydrologic information and data; adequate water supply for experimental purposes including reclaimed waters; possibility of control of pumping by others during period of investigation; availability of right of way without excessive cost; area representative of other areas in the state which are or may be threatened by sea water intrusion; and previous or current studies in the area applicable to the problem under investigation. Concludes with a proposed investigational program which is to include laboratory research, applied field experimental program, study of feasibility of subsurface barriers, and analysis of economic considerations.

Banks, H.O., Gleason, G.B., and Richter, R.C., Sea-water intrusion into ground water basins bordering the California coast and inland bays. Report No. 1, Water Pollution Investigations, Div. of Water Resources, State of California, 23 pp., Sacramento, 1950.

Presents list of prior reports and investigations relating to sea-water intrusion in California; geologic and hydrologic conditions under which sea-water intrusion may occur; and areas of known, threatened, and potential sea-water intrusion. Describes possible methods of restraint of sea-water intrusion, including: raising of ground water levels above sea level by reduction or rearrangement of pumping draft pattern, direct recharge of overdrawn aquifers to maintain ground water levels at or above sea level, maintenance of a fresh-water ridge above sea level along the coast, development of a pumping trough adjacent to the coast, and construction of artificial subsurface dikes. Summarizes present water law in California as it relates to prevention of sea-water intrusion.

Banks, H.O. and Lawrence, J.H., Water quality problems in California. Trans. Am. Geophysical Union. vol. 34, no. 1, pp. 58-66. 1952.

Paper discusses a few of the many serious and complex problems confronting water users in California. Water quality problems grouped under conditions of pollution caused by sewage and industrial waste disposal, degradation caused by man's development and use of water itself. Minor quality of water problems resulting from natural causes are recognized and discussed. Includes a section on "adverse salt balance," in which attention is called to the danger of underground water basins as reservoirs of fresh water being seriously affected. One protection against an adverse salt balance would be procured through drawing down water levels in dry periods to provide space for conservation of high quality flood waters.

Barksdale, H.C., The contamination of ground water by salt water near Parlin, New Jersey, Trans. Amer. Geophys. Union, pp. 471-474, 1940.

Describes progress of salt water encroachment into pressure aquifer. At some points penetration has progressed two miles inland from source and rate of movement was one mile in six years. Records from test wells indicate salt water advances in waves of high salinity followed by lower salinity, with each successive crest being higher as the pumping continues.

Barksdale, H.C., Ground water problems in New Jersey, Journal Amer. Water Works Assoc., vol. 37, no. 6, pp. 563-568, 1945.

Mentions briefly salt water intrusion in Atlantic City area caused by heavy pumping lowering ground water head from 25 ft. above mean sea level to 75-100 ft. below mean sea level.

Braithwaite, F., On the infiltration of salt water into the springs of wells under London and Liverpool. Proceedings Inst. of Civil Engineers, London, vol. 14, pp. 507-523. 1855.

Of historical interest only, this paper contains some of the earliest references to the problem of sea water intrusion. Describes increasing salinity of water pumped from wells in London and Liverpool and suggests that the source is from infiltrating sea water caused by the ground water table being lowered below sea level. Considerable discussion accompanies the paper on the pros and cons of this then controversial sea water intrusion theory.

Brenneke, A.M., Control of salt water intrusion in Texas, Journal Amer. Water Works Assoc., vol. 37, no. 6, pp. 579-584, 1945.

Describes proposed legislation to handle salt water disposal from oil field developments.

Brown, J.S., A study of coastal ground water with special reference to Connecticut, Water Supply Paper 537, U.S. Geological Survey, Washington, D.C., 101 pp., 1925.

Discusses source, movement, occurrence, development, and uses of ground water along the Connecticut coast. Excellent summaries are given of Badon Ghyben-Herzberg theory, fresh-salt water investigations on Holland coast, and Belgian laboratory experiments on coastal ground water by d'Andrimont. Measurements of numerous shallow wells along coast indicate contamination is restricted in all cases to a narrow zone of not more than 250 feet inland from the shore. Contamination in deep wells was found in zone up to 700 feet from shore. Indicates that ratio of depth of well below sea level to distance from shore should not exceed 1 in order to avoid pumping salt water. Fresh water was found more than 1000 feet seaward from shore in artesian zone. The zone of diffusion, based on findings from European studies, is reported to be very narrow (60-100 feet wide), becoming narrower where ground water movement is more rapid. Laboratory experiments by d'Andrimont showed zone of diffusion to be "very inconsiderable" and "quite sharp". The cone of salt water induced by pumping overlying fresh water is described and illustrated. Seasonal variation of salinity in ground water is roughly correlated to yearly temperature curve, disregarding changes caused by precipitation. Paper concludes with detailed descriptions of 186 wells investigated and comprehensive annotated bibliography.

Brown, R.H., and Parker, G.G., Salt water encroachment in limestone at Silver Bluff, Miami, Florida, Economic Geology, vol. 40, no. 4, pp. 235-262, 1945.

Describes salt water encroachment in southern Florida caused by lowered water table resulting from construction of drainage canals.

Salt water has penetrated 8000-9000 feet inland. A balance between fresh and salt water appears to be established in accordance with the Ghyben-Herzberg principle between the shore and 2500 feet inland, but beyond this the actual contact between fresh and salt water is lower than the theoretical contact. Suggests that a balance has not been reached in this latter zone because of insufficient time since the water table was lowered by drainage. The salt wedge should finally come to rest where a sufficient weight of fresh water above mean sea level will force the salt water to the bottom of the highly permeable aquifer.

Cederstrom, D.J., Chloride in ground water in the Coastal Plain of Virginia, Virginia Geological Survey, Bull. 58, 36 pp., 1943.

Primarily concerned with the occurrence of salty connate waters in inland artesian aquifers. Mentions briefly the possibilities of contamination by sea water of shallow wells along the coast. Recommends coastal wells be located a mile or more inland from the coast and be of sufficient number so that no large yields of individual wells are required.

Cross, W.P. and Love, S.K., Ground water in Southeastern Florida, Journal Amer. Water Works Assoc., vol. 34, no. 4, pp. 490-501, 1942.

Mentions sea water intrusion in the Miami area caused either by uncontrolled canals admitting sea water or by underground aquifer penetration by salt water. [illegible]

Dubois, E., Etudes sur les eaux souterrains des Pays-Bas, (Studies of the ground water of the Netherlands), Musee Teyler Archives, 2nd. ser., vol. 9, pp. 1-96, Haarlem, 1905.

Contains the results of an elaborate investigation of the water levels, movements of water, and quality of water in the lowlands of Holland. Dubois refers to the work of Badon Ghyben and Herzberg but points out that peculiar topographic and geologic features in Holland modify the application of the law of equilibrium. His observations of piezometric levels in many wells ranging from 25 to 40 meters in depth show that there is a strong flow of fresh water from the dunes toward the polders. Many data are given to show that the salinity of ground water increases with depth. Increase of salinity due to pumping also is noted. (Abstract from Brown, 1925).

Garrett, A.A., Status of salt water contamination in the coast-part of Orange County, California, as of 1950, U.S. Geological Survey, Ground Water Branch, Mimeographed Report, April, 1951.

Not available.

Gregor, H.F., The Southern California water problem in the Oxnard area, Geographical Review, vol. 42, no. 1, pp. 16-36, 1952.

Describes the importance of the overdraft and salt water intrusion problems to the economy of the Oxnard area. Illustrations show the increase in irrigated land from 1920 to 1949, the watersheds and underground reservoirs, and contour maps of water table elevations for 1931 and 1949. Mentions the common possible solutions to the intrusion problem and stresses the necessity for immediate federal action to avert further contamination of agricultural lands.

Hayami, S., On the saline disaster and variation of coastal underground water by land subsidence accompanying the great earthquake of December 21, 1947, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951, (unpublished).

After the great earthquake - epicenter 135.7E, 33.0N - on December 21, 1947, a relative land subsidence was observed along the coasts of Kii peninsula, Shikoku, and Honshu, facing the Inland Sea. The sea water gradually infiltrated into coastal low land up to the surface and caused there a grave saline disaster, especially to paddy fields. In order to study the process of infiltration, systematic observations of the distribution of salinity and variation in the piezometric height of the underground water were carried out in the village of Kawauti, situated along the coast of Shikoku where the phenomena seemed to be typical. The results of observations which indicated remarkable features in many respects are mentioned briefly and a mathematical theory of the process of infiltration is developed from them.

Herzberg, B., Die Wasserversorgung einiger Nordseebader (The water supply on parts of the North Sea coast), Jour. Gasbeleuchtung and Wasserversorgung, Jahrg. 44, Munich, 1901.

Based on a study of water level fluctuations and salinity in wells, the theory that fresh water floats on top of sea water, because of the differences in specific gravity, is deduced. The ratio of the specific gravity of fresh water to the difference in specific gravities of fresh and salt water is about 37, hence theoretically for one foot of fresh water above mean sea level there should be 37 feet of fresh water below mean sea level before salt water is reached. States that this relationship does not hold exactly in all cases, as it is strongly influenced by the fineness or coarseness of the dune sands, but that is proved to be approximately correct in numerous borings in the area. Found that water levels rise and fall with the tide, but they lag 3 to 4 hours behind the tide. Mentions that salinity of the ground water increased during a dry season and during periods of heavy pumping. (Abstract from Brown, 1925).

Horton, C.W., Salt diffusion in Woodbine sand waters, East Texas, Bull. Amer. Assoc. Petroleum Geologists, vol. 28, no. 11, pp. 1635-1641, 1944.

Presents differential equation of diffusion of salt from salt dome into ground water aquifer. Suggests convection currents exist in aquifer in order to account for the greater diffusion rate found to exist. Salt concentration in these aquifers is considerably greater than that of sea water.

Imbeaux, E., Les nappes aquiferes au bord de la mer, salure de leurs eaux, (Aquifers near the sea, Salinity of their water), Societe des Sciences de Nancy, 3rd, ser., vol. 6, pp. 131-143, 1905.

Contains a general discussion of the theory of coastal ground water based on information published at that time. (Abstract from Brown, 1925).

Isaacs, J.D. and Bascom, W.N., Water-table elevations in some Pacific coast beaches, Trans. Amer. Geophys. Union, vol. 30, no. 2, pp. 293-294, 1949.

Describes measurements of beach profiles, depth to ground water, temperature, and specific gravity of 10 beaches on Pacific coast of United States. Typical profiles are shown of two locations. Indicates that ground water table is concave upward during high tide and concave downward during low tide. Tides are found to affect ground water in only a narrow area of the beach face. In locations where sand dunes existed between the beach face and backshore, no rises in water table under the dune crests were found, in spite of extensive rains immediately preceding measurements in some instances.

Keilhack, K., Die Beziehung des Susswassers zum Saltwassers in durchlassigen Kustengebieten (The relation of fresh to salt water in pervious coastal areas), Lehrbuch der Grundwasser- und Quellenkunde, 3rd. ed., Gebruder Borntraeger, Berlin, pp. 133-141, 1935.

Presents a general discussion of fresh and salt waters in coastal aquifers. Describes and illustrates the Ghyben-Herzberg principle. Mentions direct relationship found between chloride content and average ground water withdrawal rate.

Kitagawa, K., Un aspect du developpement des etudes des eaux souterraines au Japan (An aspect of the development of ground water studies in Japan), Japanese Journal of Astronomy and Geophysics, vol. 17, no. 1, pp. 141-155, 1939.

Describes some of the ground water studies which have developed in Japan. Three of the studies mentioned are concerned with

sea water intrusion. The first of these derives parabolic equations for the fresh-salt interface, and compares them with equations developed by model studies. The three examples shown, based on different salt water densities, all show good agreement. The second study generalizes the equations of the first study. The third study verifies the shape of the interface postulated in the two previous studies by use of dyes and observation wells along a coastal section.

Lindgren, W., The water resources of Molokai, Water Supply Paper 77, U. S. Geological Survey, pp. 26-47, 1903.

Describes the geologic and hydrologic situation with regard to the ground water of Molokai, one of the Hawaiian Islands. Fresh water overlies a zone of brackish water which in turn overlies sea water along the porous coastal area. The surface of salt water is about 160 feet below ground surface along the south shore. Numerous data on wells and springs are included together with their chloride contents. A good example of the increase in salinity with pumping is shown by a 56-foot well at Kawela. The well was pumped for 30 days at a rate of 3.87 cfs. During this period the water level lowered 8 feet and the chloride content increased from 325 ppm to 1096 ppm.

Love, S.K., Cation-exchange in ground water contaminated with sea water near Miami, Florida, Trans. American Geophysical Union, Pt. 6, pp. 951-955, 1944.

Presents analyses of contaminated ground water, caused by sea water intrusion, in the Miami area. Most recent samples show chloride concentrations up to 16,500 ppm. Describes changes in the chemical character of the water and presents analyses to show that the changes are the result of cation-exchange which takes place between the magnesium and sodium ions in the water mixtures and the calcium ions adsorbed on the clay or organiz colloids in the ground. With sea water intrusion the active colloids give up calcium ions in exchange for the magnesium and sodium ions in the sea water. The resulting concentrations of cations in the contaminated water tend to be higher in calcium and lower in magnesium and sodium than would be expected from simple mixtures of ground water and sea water.

Matson, G. C. and Sanford, S., Geology and ground waters of Florida, U.S. Geological Survey, Water Supply Paper 319, p. 261, 1913.

Mentions the occurrence of salt water under fresh water in island and coastal areas. Where the aquifer consists of open limestone formations, salt water may exist upwards to about tide level. States that the fresh water overlying the salt water moves seaward by reason of its superior position. Under these conditions the thinner the fresh-water sheet the less its seaward gradient and consequently the greater the admixture of salt with fresh water.

Concludes that the chances of finding potable water are better in the sands of beach ridges and on sandy inlets than in the open-textured limestones on the keys or near the coast of the mainland. Presents data showing increase of chloride content from 17,000 ppm at 38 feet to 26,000 ppm at 495 feet at Marathon, Key Vaca, Florida.

Mazure, J.P., Enkele vergelijkende berekeningen betreffende de gevolgen van boven- and diepwater onttrekking in het duingebied (Some comparative calculations relative to the effects of deriving water from the upper and lower water-strata in the dune areas), Water, vol. 27, no. 13, The Hague, 1943.

Presents some comparative calculations for schematic cases of deriving water from upper and lower strata in the dunes. The phreatic surface will be lowered 2 meters when an amount equal to 45 per cent of the annual rainfall replenishment is pumped from the upper strata, and the phreatic surface will be lowered 0.8 meter under similar conditions when the lower strata is pumped. If water is taken from the upper strata, the fresh water volume will shrink and the brackish water will rise from 100 meters below sea level to 70-80 meters below sea level. Using water from the deeper strata, the boundary will rise to 60 meters below sea level. Fresh water tongues on both sides of the dune ridge area will shrink from 2 km. to 1.1 km when pumping the upper strata, and the tongues will eventually disappear when pumping the lower strata. These effects will only be appreciable, however, after some 100 years of pumping. Pumping from the upper strata will allow a greater part of the rainfall replenishment to be utilized than from the lower strata.

Ohrt, F., Water development and salt water intrusion on Pacific Islands, Journal Amer. Water Works Assoc., vol. 39, no. 10, pp. 979-988, 1947.

Describes and illustrates Ghyben-Herzberg principle and its application to ground water conditions on the Hawaiian and Marianas Islands. Concludes that Pacific island water supplies are very limited, depend upon the extent and stability of Ghyben-Herzberg lens, and can best be developed under given conditions by use of extensive skimming tunnels to secure required draft with the least possible drawdown.

Parker, G.G., Salt water encroachment in Southern Florida, Journal Amer. Water Works Assoc., vol. 37, no. 6, pp. 526-542, 1945.

Describes sea water intrusion in Southern Florida and presents numerous data on salinity of well samples and diffusion patterns of sea water. Principal contamination has resulted from sea water entering canals and diffusing downward and laterally into fresh water aquifers. Some contamination has also been found from sea water encroachment directly into aquifers along coasts. Studies show diffusion zone to be about 60 feet wide. Concludes that contamination is continuing and remedial measures will be necessary soon to prevent irreparable damage.

Parker, G.G., Geologic and hydrologic factors in the perennial yield of the Biscayne aquifer, Journal American Water Works Association, vol. 43, no. 10, pp. 817-835, 1951.

Mentions problem of salt water encroachment in the Miami area caused by installation of drainage canals. These canals have served to drain off fresh water in the coastal aquifers, allowed fresh water to enter inland and to penetrate aquifers, and to promote sea water intrusion according to the Ghyben-Herzberg principle by the loss of fresh water head. Maps show extent of intrusion over a 47-year period. Rate of encroachment until 1943 approximated 235 feet per year, but during the drought of 1943-1946, the rate increased to about 890 feet per year. To prevent further intrusion, low level removable dams were installed in certain canals. These dams, together with the increased rainfall since 1946, have caused a seaward retreat of the inland ends of salt water by 1950. In contrast with this, areas where no dams were installed suffered further salt water contamination.

Patrick, D.A., Ground water adjacent to four Pacific Ocean beaches, Tech. Rep. 155-35, Inst. of Engr. Research, University of California, Berkeley, 6 pp. plus 9 figs., 1950.

Describes results of four surveys of Pacific coast beaches, three in Oregon and one in California, involving determinations of chloride concentration of ground water at varying distances inland from the beach. Based on limited data, it is concluded that water behind most ocean beaches is less saline than sea water. Also, because the diffusion zone between sea water and fresh water appears to be narrow, potable ground water (less than 300 ppm Cl) should generally be found within a few hundred feet of the beach face. In localities where depressions occur back of the beach area, such as lagoons or streams, the salinity of ground water tends to increase as these surface supplies are approached.

Pennink, J.M.K., Investigations for ground-water supplies, Trans. Amer. Soc. of Civil Engineers, vol. 54-D, pp. 169-181, 1905.

Presents results of investigations of fresh and salt ground waters in the coastal dune area at Amsterdam. A geologic cross-section of the area is shown accompanied by effective sizes and uniformity coefficients encountered in the various strata. Comprehensive hydrologic study of the area is indicated by a cross-section of the area showing the fresh-salt water boundary, the directions of ground water movement in various aquifers, and the hydraulic grade lines observed in each of the aquifers. Isochlors drawn for the cross-section agree substantially with the Ghyben-Herzberg principle of fresh water floating on sea water. Data and a diagram show how salt water may be sucked into a well even though the bottom of the screen is above the original salt water level.

Poland, J.F., Saline contamination of coastal ground water in Southern California, Western City, vol. 19, pp. 46, 48, 50, October, 1943.

Describes briefly the geologic aspects of the coastal plain area in Los Angeles and Orange Counties in relation to the continued lowering of the ground water table and the consequent intrusion of sea water.

Poland, J.F., Summary statement of ground-water conditions along the coast of Orange County, California, Directors of Orange County Water District, 20 pp. plus 11 pl., 1947.

Includes comprehensive discussion of geology of the area, ground water recharge and discharge, history of decline of water levels, and saline contamination in the Santa Ana Gap and the Huntington Beach Mesa areas. Presents details of the effectiveness of the Newport-Inglewood fault in restraining salt water encroachment based on simultaneous water-level fluctuations in wells on opposite sides of known or inferred fault lines. Summarizes regions as follows:

(a) Alamitos Gap - no barrier in Recent age deposits; watertight barrier in Pleistocene deposits up to 15-20 foot heads.
(b) Landing Hill - leak in Pleistocene deposits barrier.
(c) Sunset Gap - no barrier in Recent age deposits; watertight barrier in Pleistocene deposits below 145 feet but leakage could occur above this depth.
(d) Bolsa Chica Mesa - watertight barrier up to heads of 15 feet.
(e) Bolsa Gap - no barrier in Recent deposits; watertight barrier in Pliestocene deposits.
(f) Huntington Beach Mesa - only partial barrier exists.
(g) Santa Ana Gap - no barrier in Recent age deposits; watertight barrier in Pleistocene deposits except in extreme eastern portion.

Suggests methods of controlling sea water intrusion as: balancing long-term basin-wide draft and replenishment; maintenance of fresh-water head above sea level inland from the saline front either by regulated draft or by artificial replenishment; dewatering through wells near but coastward from the saline front; and construction of impervious subsurface dikes.

Revelle, R., Criteria for recognition of sea water in ground-waters, Trans. Amer. Geophysics Union, Pt. III, pp. 593-597, 1941.

Describes three types of modification which sea water can undergo on passing through porous media. First method affects the proportion of positively charged ions, caused by base exchange between the water and minerals in the soil. Second method affects the proportion of negatively charged ions, caused by changes in sulphate and bicarbonate content due to processes of sulphate reduction and substitution of carbonic or other weak acid radicals. The third method

includes changes in both cations and anions, caused by processes of solution and precipitation which affect the relative amounts of both positive and negative ions. States that chloride is the only major ion not affected by the above processes and hence is an indicator of sea water intrusion. However, to eliminate the effect of a temporary increase in total dissolved salts, the ratio of chloride to bicarbonate is suggested. This is believed to be the most reliable indicator of sea water, although continued evaporation or sewage contamination can also cause an increase in the ratio.

Ribbius, C.E.P., Deduin water theorie in verband met de verdeeling van het zoete en zoute water in den ondergrond onzer zee duiner. I, Het drijvende duin watereiland (The dune-water theory in regard to the separation of fresh and salt ground water in our coastal dunes. I, The floating island of dune water), De Ingenieur, vol. 18, no. 15, The Hague, p. 245, 1903.

Not available.

Riddel, J. O., Excluding salt water from island wells - a theory of the occurrence of ground water based on experience at Nassau, Bahama Islands, Civil Engineering, vol. 3, no. 7, pp. 383-385, 1933.

Describes fresh and salt ground water occurrences on New Providence Island, Bahama Islands. Salt water is found 25-30 feet below the fresh water level, which is located near sea level. Salt water cones form below pumping wells and contaminate fresh water if pumped excessively. Concludes that amount of water available from ground water storage is determined by rainfall and should not be exceeded. Suggests many wells with small individual pumping drafts are better than one well with a large draft because fresh water must be skimmed uniformly off the underlying salt water.

Seno, K., On the ground water near the sea shore, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951 (Unpublished).

Near the sea shore, underground water touches brine water. Investigations and studies are reported. Herzberg and Nomitsu studied its nature, assuming that on their boundary surface the pressure of fresh water balances that of brine water, which is derived from the sea water column up to the sea level, above that level. Thus, the pressure gradient is nowhere in brine water. With some data on Japanese and Holland coasts the statistical pressures of ground water are computed. It is found that there are pressure gradients even in brine water, which shows that brine water moves. Some mathematical treatments are deduced concerning the motion of brine water.

Simpson, T.R., Salinas basin investigation, Calif. Div. of Water Resources, Bull. 52, Sacramento, Calif., pp. 138-139, 1946.

Describes extent and movement of sea water intrusion into Salinas Basin in 180-foot pressure aquifer. As of October 1945, there

were approximately 6000 acres contaminated by sea water extending up to 1-3/4 mi. inland. Rate of encroachment was 600 feet per year from August, 1944 to August, 1945. States that maximum distance of intrusion is to most inland position of trough in pressure surface under conditions of heaviest draft, estimated to include some 9200 acres. The small difference in head due to differences in specific gravity of fresh and salt water would have a negligible effect on the distance of encroachment.

Spear, W.E., Long Island sources of an additional supply of water for the city of New York, vol. 1, New York Board of Water Supply, New York, pp. 144-149, 1912.

Describes increase in chloride content of water taken from wells in Brooklyn. At one pumping station wells tapping a confined aquifer below 125 feet from the surface pumped water containing 4.5 ppm chloride in 1897. This concentration increased gradually to 500 ppm in 1902 although pumping rates had been reduced. These wells were then abandoned and water thereafter was obtained from new shallow wells. It was found that the original freshness of the water was not restored at once by shutting down pumping. The only wells not affected by brackish water were those located sufficiently far inland and tapping a sufficiently high water table. No brackish water has been found in wells located more than 2000 feet from the coast. Presents a graph, based on the Ghyben-Herzberg principle, showing the depth of wells below sea level which are salt-free plotted against the head of fresh water above sea level.

Swartz, J.H., Resistivity-studies of some salt-water boundaries in the Hawaiian Islands, Trans. Amer. Geophys. Union, Pt. II, pp. 387-393, 1937.

Describes fresh water lens floating on salt water in basal rocks of Hawaiian Islands in accordance with Ghyben-Herzberg principle. Resistivity-measurements were made to locate depths to sea water using the Lee partitioning method. This method consists of passing an electric current into the ground between two iron stakes driven into the surface and measuring potential differences with a potentiometer. On a plot of resistivity versus depth a downbreak (clockwise rotation of the curve-slope) indicates a lower resistivity, which is usually associated with the presence of salt water. A good check on the accuracy of the method was obtained where a drill-hole was placed close to a resistivity measurement. The resistivity-measurement indicated a depth to sea water of 135.2 ± 1.0 feet, while the drill-hole samples analyzed showed salinity approximating sea water at 134.4 feet. The width of zone of mixture of fresh and salt water was only 22 feet in this case, but the zone may be larger in pumping areas. Shows that depth of fresh water above sea level is a good estimate of depth to sea water based on Ghyben-Herzberg principle. Concluded that resistivity-measurement is a feasible method of determining depths to sea water and testing applications of the Ghyben-Herzberg principle.

Tolman, C.F., and Poland, J.F., Ground-water, salt-water infiltration, and ground-surface recession in Santa Clara Valley, Santa Clara County, California, Trans. Amer. Geophys. Union, Pt. I, pp. 23-35, 1940.

Describes salt-water contamination around Santa Clara Valley and shows limits of intrusion. Indicates that most contamination resulted from sea water entering abandoned, uncapped artesian wells located in tidal flats.

Toyohara, Y., A study on the coastal ground water at Yumigahama, Tottori, Memoirs of the College of Science, Kyoto Imperial University, Series A, vol. 18, no. 5, pp. 295-309, 1935.

Describes an extensive series of observations on a selected cross-section of coast to verify theoretical and model studies of fresh-salt ground waters (see Nomitsu, Toyohara, and Kamimoto, 1927). Bore holes were located at intervals inland from the shore and observations were made of ground water salinity, temperature, and depth. Several tables and graphs summarize the observational program. The results of the study show that in general the prototype conditions follow those based on theory and models; however, differences were noted due to diffusion of salt near the contact surface, unusual salt distributions in a semi-pervious stratum, and the effects of non-uniformity of sand strata.

Turner, S.F. and Foster, M.D., A study of salt-water encroachment in the Galveston area, Trans. Amer. Geophys. Union, pp. 432-435, 1934.

Presents geologic cross-sections along a 50-mile line extending inland from Galveston to Houston. Ground water occurs in artesian aquifers extending into the Gulf of Mexico. Shows cross-section of chloride-content of ground water which indicates sea water intrusion is occurring below 1000 feet in depth up to 20 miles inland. Analysis does not show rate of intrusion, but abandonment of water supply wells by Galveston in 1896 indicates encroachment is continuing.

Van Ertborn, O., Quelques mots au sujet de l'hydrologie de la cote belge (A few words on the subject of the hydrology of the Belgian coast), Bulletin de la Societe Belge de Geologie de Paleontologie et d'Hydrologie, vol. 16, pp. 517-521, 1902.

Discussion of d'Andrimont's first paper (1903, which see) on coastal ground water relations and expression of disagreement on several points.

Van Ertborn, O., La question des eaux alimentaires dans les regions dunale et polderienne du littoral belge (The question of water supply in the dune and polder areas of the Belgian coast), Bulletin de la Societe Belge de Geologie de Paleontologie et d'Hydrologie, vol. 17, pp. 297-315, 1903.

Presents further arguments against the ideas of d'Andrimont in Holland on the availability of fresh ground water in coastal dune areas. An extensive table is included listing 148 coastal localities which contain 268,000 people and the quality of water derived from wells in these localities. From the limited data available, the author points out that most of the waters found in the dunes are of poor quality and generally unpotable.

Wentworth, C.K., Specific gravity of sea-water and the Ghyben-Herzberg ratio in Hawaii, Trans. Amer. Geophys. Union, Pt. IV, pp. 690-692, 1939.

Discusses Ghyben-Herzberg ratio and importance of specific gravity of sea water in determining depth of fresh water lens. Stresses relation of ground and sea temperatures to measurement of specific gravity. States that expansion of sea water with rise in temperature takes place at a rate approximately one-fifth greater than that for fresh water through the range from 20° to 30° C. From 104 sea water samples collected, an average specific gravity of 1.02610 was found. Shows relations of ocean temperature and density with depth and of Ghyben-Herzberg ratio with depth of balance. Concludes that a value of 38 is the best single whole number to use for the Ghyben-Herzberg ratio.

Wentworth, C.K., Storage consequences of the Ghyben-Herzberg theory, Trans. Amer. Geophys. Union, Pt. II, pp. 683-693, 1942.

Describes in some detail the Ghyben-Herzberg principle and emphasizes the fact that the ratio of heads holds equally well for the water stored above and below sea level. The growth and shrinkage of the fresh water lens are discussed together with the time variations involved. Four hypothetical cases are presented to show the complexities, inter-relations, and time-variations of head, head lag, bottom equivalent head infiltration, natural and artificial leakage, draft, and changes in top and bottom storage. Concludes that top storage is the independent or leading element, whereas bottom storage is the dependent or following element, and that the specific total permeability of the bottom storage is the chief unknown factor in the entire process.

Wentworth, C.K., The process and progress of salt water encroachment, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951 (Unpublished).

Analyzes the balance between fresh ground water and adjacent salt water as a unit mechanism having a variety of forms and dimensions. The Ghyben-Herzberg system is defined and the history of understanding briefly sketched. Emphasis is given to the fact that if fresh water

in Ghyben-Herzberg balance with salt water is withdrawn artificially there must logically be a trend toward salt water encroachment. The varying geometric forms of the hydrologic mechanism are described and some of the limiting dimensions set forth. According to available data, a particular system may suffer salt water encroachment so rapidly that it is interpreted as an original condition; on the other hand, there may be a history of several decades before the encroachment is destructive at some of the accessible discharge points. Brief sketches of history and present condition of Ghyben-Herzberg systems are given for several of the better known areas, such as the Netherlands, Hawaii, New Jersey, Florida, and Pacific Islands.

Whitaker, W., The water supply of Hampshire, Memoirs of the Geological Survey, England and Wales, pp. 48-51, 1910.

Describes cases of salt water occurrences in wells located along the coastal area. Concludes that heavy pumping is the cause of sea water infiltration.

Whitaker, W., The water supply of Essex from underground sources, Memoirs of the Geological Survey, England and Wales, pp. 24-34, 1916.

Contains a lengthy discussion of saline waters, and especially of the chemical composition of mixtures of ground water and sea water and the effect of percolation through rocks. (Abstract from Water Supply Paper 537).

Zander, G. and Gleason, G.B., South Coastal Basin Investigation - overdraft on ground water basins, Bull. 53, Div. of Water Resources, State of California, 256 pp., Sacramento, 1947.

Presents detailed analyses of ground water conditions in numerous ground water basins of the South Coastal Basin. Indicates an excess of 25,000 acre-ft. exists in Los Angeles River Basin above the Narrows, and overdrafts amounting to 10,000, 21,000, and 14,000 acre-ft. exist in San Gabriel Basin, Chino Basin Group, and remainder of Santa Ana River System, respectively.

Anonymous, Characteristics and quality of water important in flooding work, <u>The Oil Weekly</u>, vol. 115, no. 6, p. 60, 1944.

Emphasizes the importance of knowing the exact chemical composition, reactions to be expected, and treatment required of water to be used for injection purposes. Items of the water analysis which may cause sand plugging are suspended matter, organic growths, iron, and manganese. Also, material and equipment corrosion may result in corrosive gases and properties in the water. States that injected water should be filtered and aerated to remove undesirable plugging characteristics, and that the pH should be greater than 7.2 (obtained by adding lime, if necessary) to prevent corrosion.

Anonymous, El Paso retards salt water encroachment in ground water by artificial recharge. <u>Engineering News Record</u>, vol. 149, no. 6, p. 62, August 7, 1952.

Results of artificial recharge of ground water at El Paso, Texas are reviewed in a U. S. Geological Survey report which is to be published soon. Studies done cooperatively with the Texas State Board of Water Engineers lead to the conclusion that present encroachment of salt water content will be retarded, and in places halted by recharging Rio Grande waters through injection wells. More than 177 mg of treated water from the river was injected into ground water reservoirs during the tests. It was possible to inject up to 6 mgd through four wells 1500 feet apart in the Montana Well Field. And the city's Mesa well field will accept recharge at an even greater rate.

Babson, E.C., Sherborne, J. E., and Jones, P.H., An experimental waterflood in a California oil field, <u>Trans. Amer. Inst. of Mining of Metallurgical Engineers</u>, vol. 160, pp. 25-33, 1945.

Describes an experimental water-flooding operation for determining whether this method offers promise as an economical one for recovering residual oil. A single injection well was drilled between old producing wells, and a water treatment plant using alum flocculation and chlorination was designed and built. Water has been injected into the input well for six months at rates in excess of 100 bbl. per day. Water is pumped from the storage basin through 3500 feet of 3-inch Transite line to the suction of a Triplex pump at the injection well. A constant pressure on this line is maintained by a back-pressure regulator, which returns excess water to the clear well. The injection rate is controlled by means of another back-pressure regulator, which by-passes water from the discharge into the inlet line to the Triplex pump. Both an orifice and a volumetric meter measure the water injected, in order to provide means of checking the injectivity characteristics of the well. Since the system is designed to operate unattended, a low-pressure cutout stops the gas engine that

drives the Triplex if the suction pressure falls. Float switches on the semitreated storage, filter, and clear-water storage operate an alarm system designed to attract the field operator if the plant fails to operate properly. The warning devices are so set that the field operator will have ample time to respond, even though he is not in the immediate vicinity of that plant. It has been found that although the water leaving the chlorinator is completely sterilized, micro-organisms reinhabit it so rapidly that the bacteria count of the injected water is many times that of the raw water. There is no definite evidence, however, that these bacterial have caused any appreciable plugging of the formation or well bore. Because it was difficult to predict the pressure required for injection (actual injection pressures varied from 296 psi to 812 psi), a Kobe Triplex pump was chosen. In operation it was found that adequate lubrication could not be obtained without contamination of the injected water. This difficulty was overcome by the use of chevron-type packing around the plunger in conjunction with gravity-feed lubrication. Since these changes have been made, the performance of the pump has been entirely satisfactory.

Bennison, E. W., Ground Water, Edward E. Johnson Co., St. Paul, Minn., pp. 486-496, 1947.

Discusses recharge wells and gives examples of their use on Long Island and at Louisville. States that theoretically recharge of a well should equal its discharge, but in practice this is frequently not true. Fundamental conditions required for recharge wells include: water put in well must be clean and free of bacteria that would cause growths in well; there must be sufficient vertical distance between the point of entrance at the surface and the water table to allow a ground water mound somewhat higher in feet than the drawdown in feet in the discharging well; and the formation opposite the screen section of a recharge well must be highly permeable and must not be cut off from the underlying water bearing formation by impervious beds. In 1937, 105 recharge wells in U.S.A. had an estimated average recharge rate of 318 gpm each.

Bliss, E. S., and C. E. Johnson. Some factors involved in ground water replenishment. Transactions Am. Geophysical Union. vol. 33, no. 4, pp. 547-558. 1952.

Lab. and field studies of water spreading on fine-texture soils are discussed. Effects on water intake rates of treatment with cotton-gin trash, grasses, detergents, and other substances are interpreted in terms of fundamental processes. The effects of soil and water properties and changes in these properties as a result of various treatments are given consideration. Microbial activity and other causes of soil sealing are discussed. Results of microbial activity and a management program that ultimately leads to an increase in water-intake rates above normal are presented.
Summary: (1) Rapid decline in water intake rate on fine textured soils associated with microbial activities as well as other factors affecting infiltration. In lab. high percolating rates maintained with sterile

soil. (2) Field ponds: High intake maintained longer under dense stands of Bermuda grass. Probably due to improvement due to microbial activity on organic residues - which leads to aggregation and stability of soil structures. Management of organic residues includes incubation and drying before benefits are achieved. Water-intake rates severely depressed during incubation. (3) 8-week incubation period gives best results.

Bradley, J. A., Discussion of "Utilization of ground water storage in stream system development", Trans. Amer. Soc. of Civil Engineers, vol. 111 pp. 330-333, 1946.

Describes and illustrates recharge wells operated by Orange County Flood Control District, California. The recharge water is delivered by gravity to spreading areas, where it is absorbed and cleared of silt. At a depth of 10 feet below the surface 6-inch concrete tile drains collect the percolating water and feed it into recharge wells. The wells are surrounded by 4-foot square redwood intake cribs to a depth of 20-feet and the tile drains penetrate these cribs. The well consists of a 12-inch steel pipe casing which is perforated in the vicinity of the drains and at various depths shown by well logs to contain pervious aquifers. A rotary motion of the water is induced as it passes through the intake perforations in the well tube to the underground strata, causing a vortex to form in the water surface of the well.

Brashears, M. L., Jr., Ground-water temperature on Long Island, New York as affected by recharge of warm water, Economic Geology, vol. 36, pp. 811-828, 1941.

States that the rate and amount of temperature rise depends upon the distribution of recharge wells and the amount and temperature of warm water returned to the ground. Recharge water has raised ground water temperatures by varying amounts, ranging from 0° to 20°F.

Brashears, M.L., Jr., Artificial recharge of ground water on Long Island New York, Economic Geology, vol. 41, no. 5, pp. 503-516, 1946.

Presents general discussion of use of recharge wells, pits, and ponds to return water to ground. Some wells are capable of returning 1000 gpm and have been in operation for over 5 years without failing. Water levels reached their lowest stage in 1941 and have recovered slowly since then, but water levels are still below sea level in many places.

Cederstrom, D.J., Artificial recharge of a brackish water well, The Commonwealth, vol. 14, no. 12, Virginia State Chamber of Commerce, pp. 31, 71-73, December, 1947.

Describes experimental test of recharging a well containing water with a chloride content of 340 ppm at Camp Peary near Williamsburg Virginia. The well is 472 feet deep and 8 inches in diameter and has

Cook 40-slot screen placed opposite medium-textured sand at 430-440 feet and 450-475 feet below surface. The well yielded 305 gpm with a drawdown of 62 feet. The water level in the well stood 70 feet below surface when recharging was commenced so that it was possible to build up a considerable head in the well. Recharging operations were begun on April 4, 1946 and a recharge rate of 250,000 gallons per day was soon reached. This decreased gradually to 200,000 gallons per day by May 17 at which time water began flowing out of the top of the casing. Water levels in nearby wells showed that recharge was not being retarded by high artesian pressure, but rather by clogging in the well itself. The well was pumped for three minutes in an effort to clear the well. The water discharged at first was fiery red but cleared by the end of the period, indicating that iron rust from delivery mains had collected on the well screen. After this clearing, however, the recharge rate was only temporarily increased, leading to the conclusion that packing of sand grains around the well screen was the primary cause of the reduced recharge rate. Recharge operations were suspended on June 28, at which time a total of 17 million gallons had been added to the well. The recharge well was pumped from July 2 to July 25 and from August 4 to September 21. Discharge began at 325,000 gallons per day and increased to 380,000 gallons per day by the end of the first pumping period. This latter discharge was also maintained throughout the second pumping period. During the first pumping period, when 8.4 million gallons were pumped, the chloride content of the discharge water increased from 10-12 ppm to 20 ppm. This indicated that with about half of the recharge water pumped out, the proportion of ground water in the water discharged was about 3 per cent. In the second pumping period, the chloride content increased to 98 ppm when a total of 12.6 million gallons had been pumped; to 220 ppm at 17 million gallons; and to 340 ppm when 25 million gallons. Summary of test results includes: about 50 per cent of the amount of water recharged was uncontaminated when pumped out, hence by allowing a portion of the first recharge water to remain in the ground most of the later recharged water can be recovered; some clogging of the recharge well is to be expected, except where the sediments are coarse-grained and where the well was highly developed when constructed; foreign material from mains should be prevented from entering the well; restoring the permeability of a clogged well is difficult - surging or the use of several wells may be necessary; and an observation well near the recharge well is of considerable value for indicating the extent of recharge and for determining whether any decrease in the rate of recharge is due to clogging or the building up of head in the area.

Cerini, W.F., Battles, W.R., and Jones, P.H., Some factors influencing the plugging characteristics of an oil-well injection water. Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 165, pp. 52-63, 1946.

A test for determining the plugging characteristics of an oil-well injection water has been developed. It consists in measuring changes of the filter rate of a water at constant pressure with cumulative through put when passing the water through a medium grade sintered glass disk. Application of the test method was made to the water used in the Union Oil Company's experimental water flood in the Richfield field, Orange County, California (see Babson, Sherborne, and Jones, 1945). This water

developed increasing plugging tendencies upon aging, even though the finished water from the plant filter exhibited few such characteristics. The plugging material was found to consist of calcium carbonate, ferric hydroxide, and organic substances that were presumed to be bacterial. Various methods of stabilizing this water to prevent the formation of plugging material were attempted. Lowering the pH to approximately 6.4 by injecting carbon dioxide provided a moderately satisfactory means for stabilization, but a better method of stabilizing this water was found to be first aging and then filtering.

Conkling, H. and Goudey, R.F., Reclaimed sewage to replace ground water in Los Angeles area, Civil Engineering, vol. 16, no. 11, p. 498, 1946.

Summarizes report to West Basin Water Association on proposed use of reclaimed sewage, which would be forced underground through wells, to solve the need for additional water supply in the area. States that treated sewage would be entirely sterile and that no feeder wells would be less than 1000 feet from pumping wells. Assumes that each recharge well could take 2 cfs, but figures only half-time operation so that an average of one cfs. would be obtained. This would require 110 wells to add 80,000 acre-feet per year to the aquifer. Estimates additional water by this method would cost only one-third as much as Colorado River water.

Dial, L.H., Disposal of salt water in the East Texas field, The Petroleum Engineer, vol. 15, no. 2, pp. 59-62, 1943.

Describes use of injection wells for disposal of salt water produced in oil production and for maintenance of underground reservoir pressure. The collecting systems, water treatment, and settling basins are described and illustrated. Presents a step-by-step procedure for development and completion of injection wells. The problem of salt water corrosion of equipment is briefly discussed.

Dickey, P.A. and Andresen, K.H., The behavior of water-input wells. Supplement 2 to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 85-109, 1947.

A comprehensive paper is presented which is concerned with the factors that affect the rate at which a water-injection well takes water. Theoretical formulas and examples of actual well behavior are given. Transient back-pressure phenomena in the reservoir are described. These phenomena have an important effect on the response of a well to changes in applied pressure. The concept of localized injectivity index, which is the conductivity of an individual input well, is introduced, and methods for its determination suggested. The factors that affect adversely the efficiency of injection wells are described and illustrations of actual well behavior are given. Among these the plugging of the sand by suspended or dissolved solids is important. A brief discussion of controlled plugging is also included. The effect of excessive pressure in causing rupture of the formations is described discussed, and illustrated by examples of such ruptures.

Dickey, P.A., et al., Increasing and maintaining injection rates of water-input wells, Supplement 2 to Secondary Recovery of Oil in the United States, American Petroleum Institute, Dallas, pp. 154-181, 1947.

In water-flooding and pressure-maintenance operations, input wells are provided for the purpose of injecting water into the oil-producing sand in order to develop or maintain pressure differentials which cause oil to move toward the producing wells. The efficiency of the input wells is, to a large measure, an indication of the efficiency of the entire flooding process. This paper, divided into four parts, discusses this subject. In Part 1, the advantages of increased injection rates of water-input wells are discussed as related to the function and behavior of such wells. Increased injection rates are desirable not only because oil production is increased, but also because of savings in operating expense and number of wells required. Part 2 is concerned with the effect of properties of flood water on rate of input and oil production. The reduction of conductivity caused by the injected water is described, particularly the problem of clays found in sandstone formations. A study is made of the various reactions of ions on the structure of clay materials and of the possible causes for the swelling of clays. Part 3 is devoted to well-input equipment and the methods of completion of water-injection wells. The advantages and limitations of various types of equipment and different methods of completion, as they relate to the problem of maintaining and increasing water-injection rates, are covered. Part 4 deals with the chemical and mechanical treatment of water-input wells. The location and nature of plugging materials are discussed. The general methods of remedial treatment - back flowing, flushing, washing, agitating, and chemical treatment - are described. and the limitations of each are given. A table and several graphs show results obtained in a number of cases.

Fancher, G.H., Theoretical calculations for use in the installation and operation of secondary-recovery projects, Supplement to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 177-181, 1944.

The present status of the calculations which apply to secondary recovery which can be made, and which are based upon the theory of fluid flow through sands, is reviewed. Because the basic formulas come from many different sources, all have been converted to practical engineering terminology and correlated in terms of practical field and laboratory units. Both water flooding and gas injection have been considered in the application of these formulas to the solution of practical problems.

Fuller, M.L., Drainage by wells, Water Supply Paper 258, U.S. Geological Survey, Washington, D.C., pp. 6-22, 1911.

States that for an effective drainage well the ground water level must be lower than the depression to be drained and that the efficiency of such wells depends upon the difference in head between the

surface and ground water, upon the porosity of the aquifer, and upon the grain of the aquifer materials. Gravel, sand, till, conglomerate and sandstone, and limestone aquifers all may be used effectively for drainage wells. For draining marshes and ponds, a bell-mouth inlet pipe is recommended; also illustrates a screened intake for protection and greater higher effective head. Mentions causes of drainage well failures as, obstruction of well mouth, clogging of aquifer around well, clogging of well screen, entrance of quicksand from aquifer, and caving of uncased wells. Suggests either heavy pumping or "steam jetting" of clogged wells as remedies. Concludes by mentioning dangers of pollution travel underground from drainage wells for industrial wastes and sewage, particularly in limestone aquifers.

Guyton, W.F., Depleted wells at Louisville recharged with city water, Water Works Engineering, vol. 98, pp. 18-20, 1945.

Because of overdraft conditions developed by large distilleries pumping in the area, water supplies of industries were shifted to the municipal supply. This eased the ground water demand, and in addition, by recharging wells during winter with city water, adequate cold water was placed in underground storage to meet heavy summer pumping requirements. Geologic cross-sections show effectiveness of recharge program in limiting overdraft. Recharge was continued for three-months in the spring of 1944, using 3 wells, into which an average total recharge rate of about 1000 gpm was maintained.

Guyton, W.F., Artificial recharge of glacial sand and gravel with filtered river water at Louisville, Kentucky, Economic Geology, vol. 41, no. 6, pp. 644-658, 1946.

Describes artificial recharge to reduce overdraft caused by heavy industrial pumping. Recharge consists of returning filtered river water to wells in winter when water temperatures are lowest. Total recharge equals approximately 2 million gpd and is successfully overcoming the previous water shortage.

Headlee, A.J.W., Interactions between interstitial and injected water - a review, Supplement 2 to Secondary Recovery of Oil in the United States. American Petroleum Institute, Dallas, pp. 148-153, 1947.

Presents the quantitative and qualitative aspects of the problem of solids forming in oil- and gas-field waters during recovery operations. The chemical reactions are considered that will influence the porosity and permeability to fluids by plugging or opening up the pores of the formation. The conditions are described under which precipitation or solution of solids takes place. The more common precipitates which form are carbonates, sulfates, sulfides, sulfur, iron oxide, silicates, and organic residues. The quantity necessary to cause serious plugging off of a well is quite

small. The extent of the plugging-off effects will depend not only on the amount of precipitate formed, but also on its nature. The particles may be too large to enter the pores of the sand, and then again, they may be small enough to circulate freely through the pores.

Horton, R.E., Drainage of ponds into drilled wells, Water Supply Paper 145, U. S. Geological Survey, Washington, D. C., pp. 30-39, 1905.

Covers same material as Fuller, 1911, (which see) and describes briefly several individual wells used for drainage in southern Michigan.

Hurst, W. and Van Everdingen, A.F., Performance of distillate reservoirs in gas cycling, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 165, pp. 36-51, 1946.

Presents formulas from which may be obtained the pressure and streamlines in a steady-state flow between input and output wells. A method is devised to compute rapidly the successive positions of a dry gas front. If the field is irregular in shape, pressure and streamlines can be obtained by a potentiometric electrical model study. The effects of different permeabilities and of the location of wells can be established by these studies. Two specific cases are discussed. The first case, considering that only "parallel flow" (flow that occurs parallel to the bedding planes of the formation) can take place, assumes that the gas-injection wells form a line source and the producers a parallel line sink. Under this assumption linear flow occurs from the source to the sink, and is independent of the positions of the two types of wells. The second case, limited to two-dimensional flow, considers a gas-input well as a point source and a producer as a point sink, thereby introducing the influence of the position of the wells in the gas recovery efficiency. From this, the composite effect of the position of the input and output wells as well as the permeability profile is obtained. Mentions that the "parallel flow" method of analysis can be used to refine estimates of the over-all recovery efficiency from water-flooding.

Jessen, F.W., Technical problems of salt-water injection, Drilling and Production Practice, American Petroleum Institute, pp. 112-121, 1944.

Presents a comprehensive discussion of the disposal of oil-field brines into subsurface formations. The engineering problems of design, selection of equipment for surface-treating facilities, completion of the disposal well, and the chemical treatment required to render the brines suitable for injection are presented.

Johnson, A.H., Ground water recharge on Long Island, Journal Amer. Water Works Assoc., vol. 40, no. 11, pp. 1159-1166, 1948.

Describes briefly the topography of Long Island, the conservation law requiring return of ground water used for industrial purposes to the aquifer, and the number of recharge wells in operation.

Illustrates primary types of recharge well construction. Most wells now installed include well screens extending below the water table, although others are used with screens either partially or entirely above water table. Gravel-packing is employed extensively. Recharge capacities per well are in the range of 100-350 gpm although some wells have been found to have rates exceeding 550 gpm.

Keplinger, C.H., Distribution systems and surface injection equipment in secondary recovery, Supplement 2 to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 51-57, 1947.

Describes distribution systems and well-injection equipment for secondary recovery of oil by air, gas, and water. Economical systems may be designed on a sound engineering basis if factors such as volumes, pressures, and expected life of proposed projects are accurately estimated. Standard surface well-injection equipment may be varied to meet local conditions. Recommends individual well meter equipment for every project.

Klaer, F.H., Jr., Artificial recharge of aquifers, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951. (Unpublished).

Discusses natural and artificial recharge. The methods of artificial recharge are classified as indirect and direct methods. Indirect methods include inducing recharge and salvaging rejected recharge by lowering ground water levels by pumping from wells and infiltration galleries. Direct methods include surface water spreading, excessive irrigation, and subsurface methods of recharging through pits, wells, and shafts. The importance of hydrologic conditions, land cover, land preparation and use, chemical, physical, and bacteriological quality of water, and well and shaft construction are considered. Problems of contamination, rise in ground water temperature, and waterlogging of land are discussed. The practice of artificial recharge throughout the United States will become increasingly important as the demands for water lead to increasing utilization of all available sources of supply.

Lane, D.A., Surface spreading-operations by the basin-method and tests on underground spreading by means of wells, Trans. Amer. Geophys. Union, vol. 15, pp. 523-527, 1934.

Describes results of spreading by means of wells in various southern California locations. In 1927 water was introduced into a group of wells in San Fernando valley having 20-in. diameters, 400-ft. depths, and pumping capacities of 5-6 sec-ft. Results were unsatisfactory - within a few days percolation rates had become so small as not to warrant continuation of the method. Redevelopment was necessary to put wells back in operation. Another well, in the Los Angeles Coastal Plain, of 16-in. diameter and 300-ft. depth, averaged a percolation rate of 116.9 gpm over

a 92-day period using an average head of 65 feet above the normal ground water table at a depth of 93 feet. A well at the Manhattan Pumping Plant, of 16-in. diameter and 625-ft. depth, was recharged starting with a head of 10 feet above the 72-ft. ground water table. Percolation equalled 0.25 AF/day and increased steadily to 0.66 AF/day when a head of 70 feet was applied. The spreading rate of this well equalled only 16.0 per cent of the pumping capacity. The recharge rate of the latter well was only 26.5 per cent of the former, due to the fact that in the Manhattan well the water-table stood up within a clay blanket, whereas in the coastal plain well the water was free to enter a dry gravel formation. Concludes that salient factors in spreading by wells are: water must be clear and free from matter which would promote bacterial growth; rates of percolation are a function of the head applied above the natural water table; and whether perforations are into a dry stratum or one containing water. Suggests that frequently surging of wells by blowing with an air-compressor will materially increase the percolation rate, and that results in one district are not necessarily applicable in another.

Laverty, F.B., Recharging wells expected to stem sea-water intrusion, Civil Engineering, vol. 22, no. 5, pp. 313-315, May 1952.

Presents a summary of the major points from the original report (Laverty, Jordan, and van der Goot, 1951), which see.

Laverty, F.B., Jordan, L.W., and van der Goot, H.A., Report on tests for the creation of fresh water barriers to prevent salinity intrusion performed in West Coastal Basin, Los Angeles County, California, Los Angeles County Flood Control District, 70 pp., 1951.

Describes well recharge tests at Manhattan Beach and El Segundo and spreading tests at Redondo Beach. Results of Manhattan Beach tests lead to the following conclusions: creating of a fresh water ridge along the coast is most promising solution to intrusion problem; bacterial slimes will form and clog aquifers being recharged unless flow is sterilized; bacterial slimes can be controlled by dosing recharge water with chlorine at an initial dosage of 15 ppm; chlorine dosages should be commenced immediately upon recharge to sterilize potential bacterial growth sources; clogging by incrustations of insoluble carbonates did not occur to a noticeable degree during tests; it is desirable to exclude air from recharge flow; recharge flow displaced saline water up to 500 ft. from the injection well; pumping tests indicated little or no mixing of fresh and salt waters; recharge well carried flow of between 1 and 2 cfs for a five-month period; pressure elevations created by recharge are approximately proportional to recharge rate; creation of a fresh water ridge at least 10 feet in height is feasible in the area; and a broader range of practical test information is essential before a criteria for well spacing can be defined. Spreading test results at Redondo Beach indicate the following conclusions: percolation rates can be maintained at 2 to 3 cfs per wetted acre; growth of soil-clogging microorganisms can be controlled by a chlorine concentration

of 3 ppm; spreading was not found feasible in areas where impervious clay strata overlay the aquifer; recommendations regarding creation of a fresh water barrier by open-basin spreading in the area cannot be made until more definite information of the underlying deposits is available; and aquifer conditions of the area indicate a supply of 60 to 90 cfs necessary to create a fresh water barrier to saline intrusion. Well recharge tests at El Segundo lead to the following conclusions: adequate percolation capacity into the confined sand dune formation can be developed by means of pit type injection wells drilled in basins or trenches and filled with pea gravel; pit type wells may be 30 inches in diameter, at least 40 feet deep, and spaced a minimum of 20 feet apart; gravel filters must be sterilized intermittently with copper sulphate, calcium hypochlorite, or other similar compounds to prevent clogging by algae or bacterial slimes; gravel-filled pit type wells cannot be cleaned when percolating capacity is lost; an 18-inch pipe extending into the well to the top of an 8-foot fill of gravel, would permit redeveloping it by pumping and surging during recharging operations; wells cost $1.20 per foot for labor, supervision, and drilling rig rental, and $.75 per foot for gravel fill, overhead not included; recharge water did not affect deeper confined aquifers below the aquifer receiving recharge water. Report includes data on the geology of the areas, locations of principal aquifers, and their permeabilities, and also plates, photographs, and maps providing detailed information on the tests conducted.

Laverty, Finley B. Ground Water Recharge, Journal American Water Works Assoc., vol. 44, no. 6, pp. 677-681. 1952.

(1) The basin method is most practical method for surface water spreading if suitable land is scarce or expensive. (2) Economics of all sources of supply in relation to the useful capacity and yield of a ground water basin should be carefully studied before comparing with cost of ground water recharge. Well recharge to reach aquifers beneath impervious strata requires considerable research before it can be said to be a uniformly successful operation. Methods are now being evolved, which indicate possible success of continuous well recharge where the objectives warrant - such as repelling sea water intrusion. Sewage reclamation for ground water recharge is indicated to be thoroughly feasible if large-scale development costs are spread over a sufficient period of time and if careful control is maintained to prevent pollution of ground water by deleterious influents.

Lee, C.H., Subterranean storage of flood water by artificial methods in San Bernardino Valley, California, Report of the Conservation Commission of the State of California, Sacramento, p. 354, 1913.

Describes use of recharge shafts in Turk Basin of Lytle Creek. Shafts are 4 ft. by 6 ft. and 50-70 ft. deep lined horizontally with 2-inch redwood planks spaced 3/4-inch apart. The 12 shafts in operation are each equipped with settling basins to remove silt in the flood waters. Reports recharge rate of 1-2 cfs for each shaft.

Leggette, R.M. and Brashears, M.L., Jr., Ground-water for air-conditioning on Long Island, New York, Trans. Amer. Geophys. Union, pp. 412-418, 1938.

Describes the use of recharge wells in returning water for industrial purposes to the ground. In 1937, 105 recharge wells were recharging at rate of 318 gpm each and the number of wells is steadily increasing. It has been found that considerable care must be given to construction details of recharge wells to obtain successful operation. General practice calls for finishing the wells below the water table, using either large diameter slotted-pipe or spiral-wound well-screens. Some wells are gravel-packed, and many subjected to a period of development. Clogging of the well-screen occurs by chemical incrustation, by mechanical clogging with silt or pipe-scale, or by a combination of both. Recharge wells discharging below water table have been more successful than discharging above water table due to three processes favoring incrustation in the latter group: pressure reduction on water leaving discharge pipe; release of dissolved gases; and water coming in contact with oxygen of the air. There should be no silt in recharge water; for example, one ounce of silt in 100 gallons of water would result in more than 11 tons of silt in a recharge well in one air-conditioning season. Discusses increases of ground water temperatures caused by recharging aquifers with warm water. Data from less than two years of records are not conclusive; some well temperatures showed no changes while others increased up to 9°F. Indicates that rate of temperature rise can best be controlled by increasing horizontal and vertical distances between supply and recharge wells.

Maxwell, N.E., Effects of permeability on secondary recovery, well spacing, and injection rates and pressures, Supplement to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 155-157, 1944.

The relation between permeability and well spacing, injection rates, and pressures is expressed in the radial-flow equation. Mention is made of modifications necessary in applying the equation to existing well spacing, of different permeabilities in different directions, and of non-uniformity of permeabilities vertically.

Meinzer, O.E., General principles of artificial ground-water recharge, Economic Geology, vol. 41, no. 3, pp. 191-201, 1946.

General discussion of artificial recharge, including indirect method by which producing wells are located as close as practicable to areas of rejected recharge or natural discharge, and direct methods such as spreading or recharge wells. Discusses the different methods in relation to geologic structure and ground-water hydraulics together with examples.

Mills, B., Injection method for salt water disposal, The Oil Weekly, vol. 97, no. 8, pp. 15-25, no. 9, pp. 19-26, and no. 10, pp. 19-27, 1940.

Presents a comprehensive description of the use of injection wells for disposal of salt water wastes from oil fields. Schematic diagrams are shown of some of the disposal plants together with numerous illustrations of the site equipment. A typical plant may include a receiving tank, settling tank, storage tanks, filter and/or aeration tanks, backwash tank, and injection wells. The cross-section of one injection well is shown. Numerous details of operation and of success of injection are included throughout the paper.

Mitchelson, A.T. and Muckel, D.C., Spreading water for storage underground, Tech. Bull. 578, U.S. Dept. of Agriculture, Washington, D.C., 80 pp., 1937.

Describes (pp. 74-76) percolation through a shaft and a well in the Lytle Creek debris core in southern California. A 4 by 6-ft. shaft was sunk to a depth of 252 feet and lined with 2 x 10 inch redwood planks spaced $1\frac{1}{2}$ inches apart. It was soon discovered that the turbulent flow washed out material and made the shaft 80 feet shallower. This surplus material was excavated, the cavities were filled with material similar to the original formation, and battens were nailed to the timbers thereby reducing the openings to $1\frac{1}{2}$ inch squares. Only clear water, measured by a rectangular weir, was supplied to the shaft. Percolation rates varied from 3.34 to 4.0 AF per day. Measurements were also made on an abandoned 16-inch well, at which percolation rates were found to vary between 1.99 AF/day and 3.69 AF/day. In both the shaft and the well, the percolation rate was less in the 1934-35 season than in the 1933-34 season.

Morris, W.S., Subsurface disposal of salt water in the East Texas field, The Petroleum Engineer, vol. 14, no. 11, pp. 41-53, 1943.

Describes how 56 injection wells in the East Texas field have disposed of more than 87,000,000 bbl. of salt water produced in oil well production. Either old oil wells or new injection wells are used for the purpose. Casings vary from $5\frac{1}{2}$ inches to 7 inches. Details of drilling, plugging, and preparing the wells for injection are presented. The salt water, containing approximately 38,000 ppm chlorides, required no treatment other than removal of oil in wells where the water was not permitted to come in contact with air. In the majority of wells, however, where an air-water contact existed, considerable treatment of the injected water was necessary. The treatment consisted of separation of the oil from the water, aeration to oxidize the iron, chlorination to kill bacteria or prevent its growth, addition of alum and lime to expedite flocculation and precipitation of foreign matter, and finally a sufficient retention time for the sedimentation process to be completed. Costs of injection, problems of chlorination, and collecting and filtering systems are also discussed.

Muskat, M. and Wyckoff, R.D., A theoretical analysis of water-flooding networks, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 107, pp. 62-76, 1934.

Discusses the theory of water-flooding networks based on the assumptions of steady state conditions, two-dimensional flow, and equal viscosities of oil and water. Typical flooding networks are shown together with corresponding pressure distributions between input and output wells. Photographs of model flood studies (see Wyckoff, Botset, and Muskat, 1933) are included. Graphical plots are presented of conductivity (production rate per unit network element per unit pressure differential), flooding efficiency, well spacing, and well density relations. The relative efficiencies of floods in the various networks are compared.

Nauth, R., Chemical aspects of water-flooding, The Petroleum Engineer, vol. 10, no. 11, pp. 21-24, 1939.

Discusses the importance of maintaining water for injection with an alkalinity of 15 ppm and a pH of 9.2. Describes and illustrates diagrammatically the treatment necessary for raw water. Mentions the problem of carbonate precipitates in the sands caused by the injected water.

Plummer, [illegible], et al., Effect of certain micro-organisms on the injection of water into sand, Petroleum Technology, vol. 7, no. 1, 13 pp., 1944.

Describes some of the common micro-organisms that occur in waters from tanks in midwestern oil-fields, the effect of these organisms on the chemical content of the waters and the bioproducts and precipitates resulting from the growth of bacteria and algae. Experimental work is presented on the effect of these organisms and their precipitates on the permeability of oil sands into which contaminated water is introduced. Application of the experiments to water-flooding and water disposal problems is indicated.

Progress Report. 1952. Artificial ground water recharge. Progress report of Task Group E4-B - Committee on Artificial Ground Water Recharge. Journal American Water Works Assoc., vol. 44, no. 8, pp. 682-684.

A survey of the extent to which artificial recharge is presently being practiced in this country. Notes that: (1) only one state requires that water taken from the ground be returned to the ground after use - N.Y. applies law to wells within Long Island. (2) Four states have recharge on a regional basis with various types of ground water supplies; ten states for a specific public water supply; in 9 states for specific industrial supply; in 8 states for a specific irrigation supply. Majority are in California or in the Eastern Seaboard States. (3) The use of treated wastes is viewed

with extreme caution. Only reported use of effluent from sewage and industrial treatment plants being used for recharge purposes is Long Island. (4) Refers to a bibliography being prepared by Fred H. Klaer and others of the U. S. Geological Survey on artificial recharge. (5) Committee plans on abstracting many of the reports listed in the bibliography.

Rhea, A.S. and Miller, E.B., Jr., Disposal of salt water in the East Texas field, Petroleum Technology, vol. 3, no. 1, 10 pp., 1940.

Describes open (water exposed to air) and closed (air excluded from water) disposal plants for disposal underground if oil-field brines. Injection wells and performance and water treatment are discussed from actual field data.

Sanford, J.H., Diffusing pits for recharging water into underground formations, Journal Amer. Water Works Assoc., vol. 30, no. 11, pp. 1755-1766, 1938.

Describes best construction of recharge wells used on Long Island. This includes a pit 30-36 inches in diameter with a center pipe 8-12 inches in diameter with an acid-resisting well screen at the bottom. Discusses cleaning methods for recharge wells. Indicates dry ice cubes give generally most satisfactory service and are very economical. Suggests hydrochloric and sulphuric acids as stronger reagents for loosening incrustations.

Scheelhaase, Dr. and Fair, G.M., Producing artificial ground water at Frankfort, Germany, Engineering News-Record, vol. 93, no. 5, pp. 174-176, 1924.

Describes operation and results of using raw river water to augment ground water supplies for Frankfort, Germany. Water taken from the River Main was first passed through a scrubber and a slow sand filter to remove turbidity and organic load. The effluent was then passed into an exfiltration gallery located 10 feet below ground surface and consisting of two 80-foot branches of open-jointed tile pipe, which were used alternately. The water table was situated 43 feet below the gallery so that 14 days were required for the water to percolate through that depth. The first observation well, located 66 feet from the gallery, showed the color, odor, taste and B-coli count of the original surface water were completely removed. The temperature of the added surface water caused a 5°F increase in summer and a 3°F. decrease in winter. No difficulties of operation were experienced and the two branches of the exfiltration gallery have not clogged during 12 years of operation.

Simpson, T.R., Recharge by percolation wells, excerpts from "Report on percolation, Feather River and tributaries, counties of Sutter and Yuba", 4 pp., 1948 (Unpublished).

Summarizes salient factors affecting recharge rate from a percolation well as: water introduced in the well must be clear and free of bacteria; recharge wells must be fully developed at the time of construction; recharge wells should be surged frequently to prevent obstruction of perforations; incrustation of perforations by bacterial growth must be prevented by use of bacteria-free water; recharge wells should not be located in areas where quicksand is encountered; and the recharge rate of a well approximates only 25 per cent of the pumping rate when the head above the water table equals the drawdown. Discusses proposed replenishment wells in Feather River streambed and concludes that 250 wells could supply 15,000 acre-feet of water in six months with a spacing of 500-1000 feet between each well. Estimates first cost at $1,000,000 for the project.

Sonderegger, A.L., Hydraulic phenomena and the effect of spreading of flood water in the San Bernardino Basin, Southern California, Trans. Amer. Soc. of Civil Engineers, vol. 82, p. 818, 1918.

Mentions use of timbered pit 5 feet by 5 feet by 40 feet deep for spreading water. Cost was $483, and admitting only clear water, absorption rate did not exceed 0.7 cfs constant flow.

Stearns, H.T., Artificial recharge of the zone of saturation, Hydrology, edited by O.E. Meinzer, McGraw-Hill, New York, pp. 697-698, 1942.

Mentions artificial recharge of ground water reservoirs composed of basalt on Hawaiian Islands. Various organizations on the Islands have made a practice of recharge aquifers through shafts and galleries to increase fresh water supply and decrease salt content. No quantitative data are available.

Stringfield, V.T., Ground water investigations in Florida, Florida State Geological Survey, Bull. 11, 33 pp., 1933.

Mentions use of drainage wells in Orlando and vicinity for disposal of sewage and runoff. Approximately 120 wells are in use, varying from 160 to 800 feet in depth and from 6 to 12 inches in diameter. The wells extend into the Ocala limestone formation and a number of them are cased. Drainage capacity of the wells ranges from less than 100 gpm to 9500 gpm. States that the effectiveness of a drainage well depends upon the permeability of the formation into which it discharges, the size and construction of the well, and the depth of the static water level below the surface intake.

Subcommittee on Secondary Recovery, Bibliography on use of water in secondary recovery, Supplement to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 245-258, 1944.

Presents a comprehensive bibliography on all phases of use of water in secondary recovery of oil (injection of water into oil sands to increase oil production). A total of 412 items are listed covering a period from 1880 to 1944.

Subcommittee on Secondary Recovery, Bibliography on use of water in secondary recovery, Supplement 2 to Secondary Recovery of Oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 205-212, 1947.

A supplement to the immediately preceding paper (which see) containing a bibliography of 169 items on the use of water in secondary recovery of oil covering the period 1943-1946.

Tolman, C.F., Ground water, McGraw-Hill Book Co., N.Y., pp. 183-187, 240-248, 1937.

Summarizes briefly results of recharge wells by Lane, 1934, (which see) and success of spreading polluted river water to recharge ground water in Germany. Describes ground water mound shapes and types - in contact and not in contact with surface water. Summarizes principles of fresh water floating on salt water from Brown, 1925 (which see).

Trauger, F.D., Description of an early experiment in ground water discharge through wells at Lindsay, California, (Unpublished report), 10 pp., 1949.

Not available.

Unklesbay, A.G. and Cooper, H.H., Artificial recharge of artesian limestone at Orlando, Florida, Economic Geology, vol. 41, no. 4, pp. 293-307 1946.

Mentions use of 175 wells in Orlando area for sewage and drainage disposal. The piezometric surface in the limestone aquifer is far enough below land surface to allow drainage by gravity. Wells seldom become clogged although considerable amounts of rubbish are carried into them. The piezometric surface of the artesian water is higher where the wells are concentrated, but the effect of the artificial recharge cannot be clearly differentiated from that of natural recharge. A deep-well current meter was used successfully to determine the horizons at which the polluted surface waters enter the limestone.

Watt, A.K., Ground water in Ontario: 1947. Ontario Department of Mines Annual Report. vol. 60 pt. 11 - 1951. Published in 1952, 116 pp.

It is a compilation of ground water data assembled from well measurements and drillers' records; source, storage, movement, methods used in recovery, and artificial recharge are included.

Yuster, S.T., Graphical prediction of water flooding intakes, The Oil Weekly, vol. 121, no. 8, pp. 36-40, 1946.

Presents a simple and rapid graphical method for the prediction of water inputs in a water flooding operation. The method is very flexible and of the four variables - effective permeability, time, cumulative volume, and injection rate - any two can be fixed and the other two can be computed graphically. Various problems are considered which illustrate the application of the graphs.

Yuster, S.T., and Calhoun, J.C., Jr., Water-injection wells and their behavior, The Oil Weekly, vol. 116, no. 3, pp. 28-37 and no. 4, pp. 44-48, 1944.

Discusses the factors which control the injection of water, their relationship, and how well behavior may be predicted. Develops equations for the total quantity and rate of input of water as functions of time. The equations are based on assumptions of simple radial encroachment and uniform permeability. The variables involved are sand thickness, fractional porosity, fractional interstitial water saturation, injection pressure, permeability, and viscosity. The equations are applied to various well patterns used in water-flooding operations in oil fields. Comparisons are made between predicted and observed water requirements for flooding. Good agreements are shown. Mentions possible errors and limitations of the equations in practice.

d'Andrimont, R., Note preliminaire sur une nouvelle method pour etudier experimentalement l'allure des nappes aquiferes dans les terrains permeables en petit (Preliminary note upon a new method of studying experimentally the phenomena of ground water in freely pervious aquifers), Soc. Geol. Belgique Annales, vol. 32, pp. M115- M-120, Liege, 1905.

Describes an experimental study using a glass vessel 60 cm square by 30 cm high filled with dune sand and tilted to reproduce the Belgian coast. Using potassium bichromate of proper density to represent sea water and uncolored fresh water to represent ground water, the flows were observed using grains of potassium permanganate. It was found that the depth to zone of contact was proportional to the height of the fresh-water surface; that the zone of diffusion was very small and sharp, that when fresh water was added to the sand, an upward movement of water was observed on the beach; that a protuberance formed in the contact zone when the upper fresh water was artificially drained; and that if an oscillation comparable to a tide was created, the contact surface oscillated also. (Abstract from Brown, 1925).

d'Andrimont, R., Sur la circulation de l'eau des nappes aquiferes contenues dans des terrains permeables en petit (On the circulation of ground water in freely pervious aquifers), Annales de la Societe Geologique Belgique, vol. 33, pp. M21-M33, 1906.

Describes in detail the investigations of Pennink, 1905 (which see), in Holland relating to laboratory studies of ground water movement. Six experiments made by Pennink are described and illustrated by photographs of the laboratory models. The first three experiments, using a glass-walled section filled with a uniform porous media and colored dye, show the flow patterns into a drainage canal. The fourth and fifth experiments, using a similar apparatus, show the flow patterns which develop beneath a well. These illustrate clearly how salt water can be drawn into a well even though the bottom of the well lies entirely in a over-riding fresh water aquifer. The sixth experiment shows the Ghyben-Herzberg principle as it might be applied to an ocean island. Thus, a cone of fresh water is found to float on salt water with the greatest elevation of the fresh water cone centered over the lowest point of salt water depression.

Bakhmeteff, B.A. and Feodoroff, N.V., Flow through granular media, Journal of Applied Mechanics, vol. 4A, pp. 97-104, 1937. Discussion, vol. 5A, pp. 85-90, 1938.

The fundamental theoretical relationships of flow through granular media are investigated based upon a series of experiments using spherical lead shot 0.038 to 0.361 inch in diameter. It was found that the Darcy type of flow was superseded by a zone where losses became an increasing power of

the velocity, until in the higher ranges a stable pattern with an exponent of 1.8 of the velocity was reached. The experimental data were reduced to a unified basis in terms of a Reynolds characteristic. The possible mechanism of resistance in the different flow forms is discussed. Concludes with a discussion of the Darcy filtration coefficient, taking into account the porosity, viscosity, and shape factor.

Baldwin - Wiseman, W.R., The flow of underground water. Proceedings of the Institution of Civil Engineers, vol. 165, pp. 309-352, 1906.

Detailed summary of research covering four main points: experiments on the rate of flow of water through moderately large blocks of stone of various thicknesses under pressures varying between atmospheric pressure and 75 lbs. per sq. in. above atmospheric pressure; experiments on the variation of hydraulic pressure within a rock at various depths from the pressed surface; experiments on the relative porosity and retentivity of rock and sand; and an investigation of the statistics of pumping and filtration plants. Concludes by presenting a generalized equation for the yield of a well based upon its physical dimensions and the permeability of the aquifer. Appendix includes summary of formulas by various investigators and numerous tables relating to the experiments and to British water supplies.

Baumann, P., Ground-water movement controlled through spreading, Proc. Amer. Soc. of Civil Engineers, vol. 77, sep. 86, 38 pp., 1951.

Presents detailed mathematical analysis of two-dimensional ground water flow in relation to the mound formed under spreading grounds in unconfined aquifers. Mentions application to control of sea water intrusion by maintaining mound height along coast equal to at least mean sea level. Gives three-dimensional flow equations for spreading grounds over horizontal and inclined aquifer. States that mound width for circular spreading ground of 500 feet radius with recharge of 1,000,000 cu. ft. per hour on aquifer of .01 slope equals 10,000 feet. Describes model setup, experiments, and results to verify theoretical flow patterns.

Botset, H.G., The electrolytic model and its application to the study of recovery problems, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 165, pp. 15-25, 1946.

It is possible by means of the electrolytic model to simulate water-flooding or gas recycling systems involving input and output wells, and also the encroachment of a natural water drive. The results are obtained pictorially, and by simple measurements and calculations the percentage recovery is obtained quantitively as a function of the total input. The underlying theory of the operation of the model is based on the fact that Ohm's law and Darcy's law are exactly analogous; hence electrical flow through a conducting medium may be used to simulate

homogeneous fluid flow through a permeable medium. In order for the analogy to hold exactly, the sand must be assumed to have uniform porosity and permeability and the input and output fluids to have identical viscosities and densities. In using the motion of ions to represent fluid flow in a sand, it is necessary, of course, to have the ions visible so that their motion may be observed. For this purpose, copper-ammonium ions, which are deep blue in color, are used to represent the input fluid, while the sand is represented by an agar gelatin solution containing colorless zinc-ammonium ions. Both types of ions have the same mobility, so that two fluids, of equal viscosity are represented. The model consists essentially of: a transforming-rectifying system for converting 110 volts of alternating current to any desired voltage of direct current; a one-foot square transparent model of the field of interest containing an agar gelatin and zinc-ammonium chloride; transparent plastic wells, $\frac{1}{2}$-inch inside diameter and $1\frac{1}{2}$ inches long, containing a copper-ammonium chloride solution and penetrating the gelatin field; and a mirror and camera arrangement to photographically record the flood, located under the table below the glass plate of the flood. A detailed description of the model apparatus is presented together with an electrical circuit diagram and a photograph of the apparatus. Discusses preparation of a field of variable thickness (involving use of a transparent plastic cast to the desired shape), operation of the model, and the model limitations.

Bruce, W.A., An electrical device for analyzing oil-reservoir behavior, Petroleum Technology, vol. 6, no. 1, 13 pp., 1943.

Presents the theory and construction of an electrical analogy model and its application to water, oil and gas flow systems in reservoirs. An example is given for an actual oil field and the analyzed and measured pressures are compared.

Dietz, D.N., Een modelproef ter bestudeering van niet-stationnaire bewegingen van grondvater (A model test for studying non-steady ground water flow), Water, vol. 25, no. 23, The Hague, 1941.

Describes a model test for two-dimensional studies of ground water flow. The model consists of two glass plates, fixed together at a distance of 0.002 meter, paraffine-oil being used as a medium. The viscous liquid will encounter a linear resistance, thus fulfilling the Darcy law. (Abstract from Krul and Liefrinck, 1946).

Dietz, D.N., Ervaringen met modelonderzoek in de hydrologie (Experiences gathered in model research for hydrologic investigations), Water, vol. 28, no. 3, The Hague, pp. 17-20, 1944.

Describes the various models used by the Netherlands Government Institute for Water Supply, and the experiences gathered in different investigations. (Abstract from Krul and Liefrinck, 1946).

Fancher, G.H. and Lewis, J.A., Flow of simple fluids through porous materials, Industrial and Engineering Chemistry, vol. 25, pp. 1139-1147, 1933.

Describes and illustrates an apparatus for measuring and studying the flow of fluids through porous media. Knowing the porosity and screen analysis, the data are used in the construction of a friction factor chart in terms of the properties of the fluids and sands. Concludes that fluid flow in porous media closely resembles that through pipes, namely viscous and turbulent flow types.

Franzini, J.B., Porosity factor for case of laminar flow through granular media, Trans. Amer. Geophys. Union, vol. 32, no. 3, pp. 443-446, 1951.

The resistance coefficient of flow through granular media depends on the porosity of the medium as well as the Reynolds number of the flow. There is disagreement as to the functional relationship between porosity and resistance coefficient, hence a series of carefully conducted permeability test was made on randomly packed and systematically packed media using a variable head permeameter. The test results indicated that the relationship between porosity and resistance coefficient for laminar flow through granular media is represented best by the Fair-Hatch expression $(1-a)^2/a^3$ when compared with other factors that appear in the literature.

Givan, C.V., Flow of water through granular materials - initial experiments with lead-shot, Trans. Amer. Geophys. Union, pp. 572-579, 1934.

Describes results of experiments measuring pressure required to force water through three sizes of pipe filled successively with three sizes of lead-shot. Results are presented in plots of friction factor times Reynolds' number (K x R) against Reynolds' number (R). Scatter of data follow theoretical straight lines very well, except for unexplained deviations found at both high and low Reynolds' numbers. Some deviations are explained by porosity changes, lack of true spherical shape of some lead-shot, and experimental errors.

Gunther, E., Untersuchung von grundwasserstromungen durch analoge stromungen zaher flussigkeiten (Investigation of ground water flows by means of analogous flows of viscous liquids), Forschung auf dem Gebiete des Ingenieurwesen, vol. 11, no. 2, pp. 76-88, 1940.

Presents the mathematical basis of model experiments of ground water flow and then describes and illustrates the model apparatus. This consists of viscous liquid flow between two closely-spaced glass plates, based on the original work of Hele-Shaw, 1899 (which see). The analogies of the Darcy law and the Hagen-Poiseuille law to the flow of viscous liquids are shown mathematically and then demonstrated by experiments. The models are used to determine flow patterns for various problems,

including: velocity distribution for flow into a slot; refraction of streamlines (shown mathematically as well as experimentally), flow to a level slot, flow through a dam, and seepage from a canal bed.

Hatch, L.P., Flow through granular media, Journal of Applied Mechanics, vol. 7A, pp. 109-112, 1940.

Presents mathematical treatment and discussion of flow through granular media based on the previous work under the same title by Bakhmeteff and Feodoroff, 1937 (which see). States that the equation of flow through granular media is essentially identical to the equation of flow through pipes, but some of the variables must be modified in order to make the expression useful. Discusses these variables and sets up formulas to determine resistance to flow through granular media. Experimental results show close agreement between calculated and observed values.

Hele-Shaw, H.S., Stream-line motion of a viscous film, Report of the 68th. Meeting of the British Association for the Advancement of Science, London, pp. 136-142, 1899.

Describes the experimental work toward the development of a model to illustrate streamline flow. The apparatus developed consisted of two parallel closely-spaced sheets of glass which are filled between with a viscous fluid such as glycerine, colored-liquid sources to reveal the flow pattern, and a lantern arrangement for projection purposes. The streamlines observed in the model were observed to agree with those found by theoretical means. Mentions also the application of the apparatus for investigating effects of variable resistance and for studying the effect of using a wedge-shaped section.

Hickox, G.H., Flow through granular materials, Trans. Amer. Geophys. Union, pp. 567-572, 1934.

Discusses the analogy between Weisback's formula for flow in pipes and viscous flow in granular media. Derivation of formula for granular media shows that identical formula can be used. Experimental results using sand are found to compare favorably with those of other investigators using lead-shot, sand, or gravel in water or oil. Better knowledge of shape-factor of materials, length of path of flow in mixtures, and relation of actual size of particles to sieve-openings are needed to obtain closer calculation of flow through granular materials.

Horner, W.L. and Bruce, W.A., Electrical-model studies of secondary recovery, Supplement to Secondary Recovery of oil in the United States - 1942, American Petroleum Institute, Dallas, pp. 190-198, 1944.

The physical and mathematical relationships of electrical models are outlined together with conversion of units from model to reservoir.

Describes the various types of models, based on electronic and ionic conduction, which have been developed, including: sheet monel (AC and DC) models, gel models, color-tracer models, and low-conductivity water models. The application of a model study to a gas cycling operation is described in detail.

Hubbert, M.K., Theory of scale models as applied to the study of geologic structures, Bull. Geol. Soc. America, vol. 48, no. 10, pp. 1456-1520, 1937.

Presents the theory of mechanical scale models as related to geologic problems including: geometric, kinematic, and dynamic similarity; forces due to various causes; relation between model ratios and physical dimensions; special cases of negligible forces, such as inertial, gravitational, and resistive; and examples. Describes various geologic structural models, including salt domes.

Hurst, W., Electrical models as an aid in visualizing flow in condensate reservoirs, The Petroleum Engineer, vol. 12, no. 10, pp. 123-129, 1941.

Presents discussion and application of electrolytic and potentiometric models as applied to gas cycling in condensate reservoirs. The theory of models is described together with illustrations of flow patterns for various arrangements of injection wells. Detailed calculations are shown for one example of three different sand strata in which the percentages of recovery of wet gas are desired.

Hurst, W. and McCarty, G.M., The application of electrical models to the study of recyling operations in gas-distillate fields, Drilling and Production Practice, American Petroleum Institute, pp. 228-240, 1941.

Presents the potentiometric method to determine the wave fronts swept out by the dry gas in displacing the wet gas from a reservoir in a recycling operation. The experimental models used are illustrated and described in detail, together with the mathematical interpretations in comparing electrical phenomena to the reservoir flow conditions.

Kelton, F.C., An electrolytic-model study of cycling in the Grapeland field, Houston County, Texas, Supplement to Secondary Recovery of Oil in the United States, American Petroleum Institute, Dallas, pp. 199-205, 1944.

Presents a comparison of electrolytic-model and field data for estimating future production rates. The model results checked very well at the beginning of the forecast period but were somewhat lower than observed values near the end of the period. This discrepancy, however, is attributed to errors in estimates of the reservoir size. Several discussions follow the paper which are concerned with model developments, applications, and limitations.

King, F.H., Principles and conditions of the movements of ground water, 19th Annual Report, Part II, U.S. Geological Survey, pp. 59-294, 1899.

Comprehensive discussion of ground water knowledge at time of publication. Topics covered include: amount of water stored in the ground; gravitational, thermal, and capillary movements of waters; experimental investigations regarding flow of fluids through porous media; and rate of flow of water through rock and sand. Numerous illustrations of laboratory equipment and data from experimental studies are included.

Krul, W.F.J.M., and Liefrinck, F.A., Recent ground water investigations in the Netherlands, Elsevier Publishing Co. Inc., New York, 78 pp., 1946.

Constitutes an excellent summary of all ground water studies in the Netherlands over the past 50 years. Because of the importance of obtaining an adequate water supply for Amsterdam and Leyden, the nearby coastal dune areas have been studied extensively with regard to the ground waters which they contain. These studies have included not only geo-hydrologic investigations of the aquifers themselves, but also have been extended to principles of ground water flow, fresh-salt ground water relations, and model studies as well. Included in this summary are the following general topics: a review of the geo-hydrologic conditions of the Netherlands; the hydrologic investigations of the dunes - fresh water- salt water relations of the dunes (Ghyben-Herzberg principle), flow patterns of rainfall entering the dune surfaces, methods of reconnaissance of the sub-soil conditions and ground water quantity and quality, and rainfall and lysimeter studies; natural and artificial ground water replenishment of the dunes (most artificial replenishment is accomplished with ponds and canals; trials with infiltration wells met with little success because of clogging and the high degree of purification required); regulation of ground water levels behind the dunes; ground water investigations for public works, such as canals, tunnels, and new polders, studies using three-dimensional sand and gravel models; and studies using two-dimensional models of two parallel glass plates spaced 2 mm. apart and filled between with a viscous liquid to assure laminar flow conditions. The book concludes with a 39-item bibliography including short abstracts of the more recent Dutch papers.

Lee, B.D., Potentiometric-model studies of fluid flow in petroleum reservoirs, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 174, pp. 41-66, 1948.

A simplification of the method of Hurst and McCarty, 1941, (which see), for conducting potentiometric model studies by the single probe method is presented along with experimentally determined invasion patterns for certain idealized flow problems. The analytical solution for one class of the problems is given. Finally, a general description is given of a newly developed instrument, (Chronocartograph) which permits direct mapping of flow lines and determination of transit times in potentiometric-model studies.

Leverett, M.C., Lewis, W.B., and True, M.E., Dimensional-model studies of oil field behavior. _Trans. Amer. Inst. of Mining and Metallurgical Engineers_, vol. 146, pp. 175-193, 1942.

States the theory underlying the design of two kinds of dimensionally scaled models of parts of idealized oil fields. One of these simulates an oil well and its surrounding sand for a distance of 16 feet radially from the well. The other model simulates linear flow through layered sands. Construction and operation of the models are described and typical data given. The unique features in the design of the models are: (1) the treatment of the permeability-viscosity quotient as a single variable and (2) the use of previously reported experimentally developed relations involving relative permeability, saturation, capillary pressure, porosity, permeability, and interfacial tension. The models at present simulate flow of oil and water only through unconsolidated sands. They are superior to unscaled models, which may give rise to erroneous conclusions. The models are designed to study the desirability of various methods of well completion and the effects of various factors on recovery of oil from layered sands.

Levine, J.S., Bissey, L.T. and Yuster, S.T., Model for study of oil recovery problems. _The Oil Weekly_, vol. 103, no. 4, pp. 34-46, 1941.

A 5-spot model has been designed and constructed for the purpose of studying the flooding of an unconsolidated sand. The apparatus is made of transparent Plexiglass so that the actual flooding patterns can be seen during various stages of the flood, and the model is adaptable to various pattern arrangements. The model consists of two Plexiglass plates 24 inches square fastened in a brass frame and spaced 1-1/8 inches apart. The space between the plates is filled with sand. Into the plates are drilled 5/16-inch holes representing wells and all of the input wells were connected by a manifold system while the producing wells were equipped with "drips" to collect produced liquid. In the experiment described the sand was saturated with oil and water was injected into the wells. A series of photographs of the model taken during flooding experiment shows the method of fluid migration from input to producing wells. Curves are given to show the actual volumes of oil and water produced.

Marshall, D.L., and Oliver, L.R., Some uses and limitations of model studies in cycling, _Trans. Amer. Inst. of Mining and Metallurgical Engineers_, vol. 174, pp. 67-87, 1948.

The use of model studies for the development of invasion patterns for cycling is illustrated by model studies obtained with a recently developed apparatus (see Lee, 1948) in the solution of actual cycling problems. Concludes that model studies are of great benefit in the design of cycling operations. The accuracy with which model studies can depict the course of cycling is limited by: (1) the accuracy with which the configuration of a reservoir and its thickness, porosity, interstitial water and permeability are known, and (2) the accuracy with which production from and injection into the reservoir are known.

Mavis, F.T. and Tsui, T.-P., Percolation and capillary movements of water through sand prisms, Bulletin 18, Univ. of Iowa Studies in Engineering, Iowa City, Iowa, 25 pp., 1939.

Presents detailed description of laboratory investigations concerning water movements through various sands. Laboratory apparatus used included a large flume, a small flume, and a capillarimeter, sketches of each are shown. The large flume was used to observe the flow of water through three gradations of sand and to observe piezometric ground water profiles; the capillarimeter was used to determine the capillary rise of water in granular materials as a function of grain size, porosity, and gradation of materials; and the small flume was used to determine directly the ground water profiles (glass-walled flume) and corresponding piezometric pressures measured at the bottom of the flume, velocity distributions through the body of the sand prisms, and movement of water in the capillary fringe. The results of the tests lead to the following conclusions: the shape of the piezometric ground water profile connecting two fixed points in a sand prism for lateral seepage flow is independent of the gradation of the material; the water in the capillary fringe moves with the gravity water in the main body of a permeable material and its average velocity is approximately two-thirds the velocity of the gravity water; the capillary rise in sand was found to be a function of the porosity of the sand and the harmonic mean diameter of the sand grains; and the piezometric profile was found to fit a slight modification of Dupuit's formula.

Muskat, M., The theory of potentiometric models, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 179, pp. 216-221, 1949.

The detailed analogy between flow systems in porous media and the corresponding potentiometric model systems is developed under conditions where it may be desirable to take into account variable thickness, variable porosity, and permeability, and also the dependence of the fluid density on pressure. It is shown that in such models it is only necessary that the electrolyte thickness be made everywhere proportional to the millidarcy-feet of the formation. The porosity does not enter directly in the construction of the model. It is introduced only in translating the electrical voltage gradient measurements into the equivalent fluid travel times. A discussion of this procedure is given.

Nomitsu, T., Toyohara, Y., and Kamimoto, R., On the contact surface of fresh- and salt-water under the ground near a sandy sea-shore, Memoirs of the College of Science, Kyoto Imperial University, Series A vol. 10, no. 7, pp. 279-302, 1927.

Based on hydrodynamic principles, the equation for a boundary surface between fresh and salt water is developed for a long straight coast and for a sandy island of circular form. Mentions that from these equations the horizontal limit of sea water percolation can be determined if the thickness of the pervious stratum is known. States that the theory

is based on an ideal case, and that the following items will cause discrepancies: non-homogeneous soil conditions, vertical motions near the boundary surface, disturbances of water near the boundary surface, diffusion of salt water, and adsorption of salt water by the sand stratum. Models for verifying the theoretical studies were constructed consisting of rectangular wooden boxes with compartments at each end partitioned off by wire gauze for fresh and salt water. The method of operation of the models, the difficulties inherent in them, and the results obtained are all presented in some detail. The experimental and theoretical contact surfaces were found to be in good agreement. The soil permeability was determined independently and found to check closely with the experimental results. The effect of diffusion on the fresh-salt water boundary was studied and it was found that its influence was negligible when the time consumed in the model experiments was considered.

Pennink, J.M.K., Over de beweging van grondwater (On the movement of ground water), De Ingenieur, no. 30, July 29, 1905.

Describes extensive laboratory experiments on the circulation of ground water, which were made by Pennink in connection with his studies of the water supply of Amsterdam. Same material is included in d'Andrimont (1906) and Pennink (1915), which see. (Abstract from Brown, 1925).

Pennink, J.M.K., Grondwater stroombanen (Motions of ground water), Amsterdam, 1915.

This book, printed in Dutch, German, French, and English, describes experiments made in 1904 and 1905 on the form of the lines of flow of ground water in pure sand. Contains details of experiments on the movement of fresh water floating on salt water and presents diagrams and photographs of the apparatus used. (Abstract from Brown, 1925).

Plummer, F.B. and Woodward, J.S., Experiments on flow of fluids through sands, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 123 pp. 120-132, 1937.

Describes experimental methods of study of linear and radial flows through consolidated and unconsolidated sands. The apparatus consists of controlled flow rates through small cylindrically-encased samples of the porous media. The apparatus used is illustrated by photographs and its operation described in detail. Other studies made include permeability determinations, effect of size of hole on rate of flow, effect of depth of penetration on rate of flow, and effect of presence of water on flow of oil through cores of oil sands. Includes extensive bibliography of 39 items on various phases of flow in porous media.

Rose, H.E., An investigation into the laws of flow of fluids through beds of granular materials, Proceedings Inst. of Mech. Engrs., vol. 153, pp. 141-148, 1945.

Presents a general equation governing the flow of liquids through a bed of granular material derived by the use of the method of dimensional analysis. The equation is verified by experiments using closely graded spherical shot and liquids having absolute viscosities ranging from 16.0 to 0.01 poises.

Santing, G., Infiltratie en modelonderzoek (Infiltration and model research), Water, vol. 35, no. 21, pp. 234-238, and no. 22, pp. 243-246, The Hague, 1951.

The hydrologic basis of infiltration, principles of model research, use of hydrologic models, and electrolytic models are discussed. The main portion of the paper concerns the principles, construction, and operation of the capillary plate model for studying fresh-salt water relationships in aquifers. These relationships in the dune areas of Holland are described and the results of the model studies are applied for interpreting the observed phenomena.

Santing, G., Modele pour l'etude des problems de l'ecoulement simultane des eaux souterraines douces et salees, (A model for the study of problems of simultaneous flow of fresh and saline ground waters), Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951 (Unpublished).

Describes a model for the study of problems of two dimensions of simultaneous flow of fresh and saline ground waters in a non-steady state. The principal part of the model, representing a vertical section of the underground aquifer to be studied, is composed of two vertical transparent plates mounted a distance apart of capillary dimensions (less than 3 mm.). In the interstice the flow is made laminar for a given liquid viscosity. The interstice represents the pores of the soil; the liquid, the ground water; and the surface of the liquid, the phreatic level. One can duplicate aquifers composed of beds of different permeabilities by the local variations of the dimensions of the interstice - the larger spacing representing beds of good permeability and the smaller spacing representing semi-pervious beds. In examining a problem of fresh and saline ground waters, one uses two liquids of approximately equal viscosities but of different densities. The scales of a model must obey certain relations which one can deduce from the differential equations of the problem to be studied. It is rarely possible, nevertheless, to fulfill all of the conditions expressed by the relations: there often remains two inevitable anomalies. The faults which result are to be noted, however, as being in general not very serious, and thus the exactitude of the results is ordinarily satisfactory. The application of the model is large when it is seen that scales can be adapted to the needs of any case. The time scale, for example, can be chosen to have a value of 1:10,000,000 as well as of 1:100.

Stearns, N.D., Laboratory tests on physical properties of water-bearing materials, Water Supply Paper 596-F, U.S. Geological Survey, pp. 121-176, 1928.

Describes in considerable detail the apparatus and methods used in making tests of mechnical composition, porosity, moisture equivalent, and permeability of water-bearing materials. Data are presented from 97 samples analyzed and interpretations of these results are made. Concludes with an outline of the works of Hazen, King and Slichter in regard to the relation between effective size and permeability.

Stevens, O.B., Electric determination of the line of seepage and the flow net of a ground water flow through joint regions with different anisotropy, De Ingenieur in Nederlandsch - Indie, Sept. 1938.

Not available.

Swearingen, J.S., Predicting wet gas recovery in recycling operations, The Oil Weekly, Vol. 96, No. 3, pp. 30-38, 1939.

Studies the problem of the area swept out by an injected dry gas to produce wet gas in cycling operations. Since the movement of high pressure gas through sand is linearly proportional to the pressure differential, the movement is similar to that for liquids or for flow of electric current through conductors. Because of this relationship, the problem was investigated using electrical models of the type described by Botset, 1946 (which see). Various arrangements of input and output wells were tried in the model and figures are presented showing the successive encroachment patterns obtained.

Vreedenburgh, C.G.J. and Stevens, O., Electrodynamisch anderzoek van potentail - stroomingen in vloeistoffen, in het bijzonder toegepast op vlakke grondwaterstroomen (Electro-dynamic investigations of potential flow in liquids, especially applied to a plane ground water flow), De Ingenieur in Nederlandsch-Indie, no. 32, 1933.

Not available.

Vreedenburgh, C.G.J. and Stevens, O., Over gronwaterstroomen en het onderzoek hunner stroomingsbeelden met behulp der electrische methode (On ground water flow and the investigation of its field of streamlines with the aid of the electrical method), De Ingenieur in Nederlandsch - Indie, no. 6, 1936.

Not available.

Wolf, A., Use of electrical models in study of secondary recovery projects The Oil and Gas Journal, vol. 46, no. 50, pp. 94-98, 1948.

Describes and pictures the potentiometric model, together with the principle of the model, its application to gas cycling, its accuracy, and other applications.

Wyckoff, R.D., Botset, H.G., and Muskat, M., Flow of liquids through porous media under the action of gravity, Physics, vol. 3, no. 2, pp. 90-113, 1932.

Describes and illustrates laboratory experiments on a radial sector of sand which show that the Dupuit formula of 1863, stating that the fluid outflow is proportional to the square of the differences in the fluid heights in the sand, is exact within experimental error. Cases where an added pressure head is superposed upon the gravity flow are investigated and the pressure distribution and fluid outflow are found to be additive. A semi-quantitative treatment is given for the flow in the capillary zone; however, in most practical cases this flow may be ignored. A theoretical discussion is given of the conditions at the free surface of a gravity flow system and it is shown that nowhere can the slope exceed 45°. The results of the radial flow experiments are generalized for broad application.

Wyckoff, R.D. and Botset, H.G., An experimental study of the motion of particles in systems of complex potential distribution, Physics, vol. 5, no. 9, pp. 265-275, 1934.

A method is described whereby the motion of the interface between two liquids of equal viscosity and density moving through a porous medium in systems of complex pressure distribution may be obtained by the use of electrolytic models. Experimental results for certain simple cases are compared with results obtained analytically and shown to check reasonably well. Illustrations are given of more complex cases showing the application of these models to various practical problems of oil field technology.

Wyckoff, R.D., Botset, H.G., and Muskat, M., The mechanics of porous flow applied to water-flooding problems, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 103, pp. 219-249, 1933.

Describes use of an electrolytic model for studying the steady state viscous flow through a horizon of uniform thickness and permeability. Assumes that the liquid flooded out, say by water from a recharge well, has the same viscosity as the flooding liquid. The effects of gravity and slope have been ignored. Numerous tests with the electrolytic model were performed, including: line drive into a single well; drive between an input well and an output well; line drive into a double line of wells; offsets staggered; line flood from two sides of a triple line of staggered wells; and various ratios of the number of input wells to the number of output wells. Many of the tests are illustrated by successive composite photographs showing the developing flow pattern. Mentions the

effects due to barriers in the flow path, to gravity both in the horizontal and inclined cases, and to viscosity differences. In the discussion following the paper, the problem of trapped gases in the aquifer is considered in relation to permeability.

Wyckoff, R.D. and Reed, D.W., Electrical conduction models for the solution of water seepage problems, Physics, vol. 6, no. 12, pp. 395-401, 1935.

Describes the construction of electrical models for solving problems of the flow of liquids through porous media under the action of gravity. With these models the shape of the free surface and the extent of the surface of seepage are determined simultaneously with the potential distribution within the flow system. Examples are given of the application of the method to the problem of water seepage through dams. The method of applying these models to the study of more complex gravity flow systems is briefly indicated.

PART V -- GROUND WATER FLOW

Dachler, R., Grundwasserstromung, (Ground water flow), J. Springer. Vienna, 1936.

Not available.

Dietz, D.N., Detoepassing van invloedsfuncties bij het berekenen van de verlaging van het grondwater tengevolge van wateronttrekking (The application of influence functions in calculating the lowering of the ground water resulting from draining), Water, vol. 27, no. 6, The Hague, 1943.

Presents a number of formulas for influence functions which have been collected by the author with a view to facilitating the calculation of the yield of wells in fields of different shapes. (Abstract from Krul and Liefrinck, 1946).

Fair, G.M. and Hatch, L.P., Fundamental factors governing the streamline flow of water through sand, Journal Amer. Water Works Assoc., vol. 25, no. 11, pp. 1551-1563, 1933.

Presents generalized equations of water flow through sand for an unstratified sand bed, a stratified sand bed, and an expanded filter bed. States that tests have been made by the authors which show close agreement between theory and experiment, but these results are not included. Mentions that the equations have application to ground water movement.

Gardiner, W., Collier, T.R., and Farr, D., Fundamental principles governing the control of ground-water, Trans. Amer. Geophys. Union, pp. 563-566, 1934.

This paper is an abstract of a proposed bulletin summarizing applications of the generalized Darcy's law to practical ground water problems. The problems briefly described include: (1) one-dimensional flow through a uniform stratum of uniform thickness; (2) flow in pressure aquifer with converging confining strata; (3) flow in pressure aquifer with parallel confining strata; (4) shape of piezometric surface in vicinity of a battery of wells arranged in a circle; (5) spacing of tile drains in a homogeneous soil overlying an artesian aquifer; (6) relation of efficiency of drainage well to effective diameter of the well; (7) maximum distance irrigation-water can "sub" from a sand stratum underlain by clay; (8) relation between minimum soil depth required to prevent erosion and rainfall-intensity on mountain slopes; (9) loss by seepage from a circular canal; and (10) rate of drying out of a stratum of soil by downward percolation into the deeper soil.

Hubbert, M.K., The theory of ground water motion, Journal of Geology, vol. 48, no. 8, pp. 785-944, 1940.

Presents a comprehensive theoretical treatment of the fundamentals of ground water flow. Treatment is based on potential at a given point being defined as equal to the work required to transform a unit of mass of fluid from an arbitrary standard state to the state at the point in question. Discusses two-fluid and three-fluid cases and application to coastal ground water conditions. States that Ghyben-Herzberg principle is incorrect because it does not take into account dynamic equilibrium between flowing fresh water and static salt water. Applies theoretical analysis to the problem of ground water changes induced by the proposed sea-level Florida ship canal.

Irmay, S., Darcy law for non-isotropic soils, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951 (Unpublished).

Extends Darcy's law, governing seepage of water through soils, to non-isotropic media. The permeability coefficient K becomes a second-degree tensor defined by nine coefficients.

Irmay, S., Formulae of steady and unsteady flow in capillary tubes with application to flow through soils (Darcy Law), Commission des Eaux Souterraines, International Union of Geodesy and Geophysic, Ninth General Assembly, Brussels, 1951, (Unpublished).

Transforms the Poiseuille formula for steady flow in capillary tubes into Darcy formula for porous media in the Kozeny form. Unsteady flow in a horizontal capillary tube gives asymptotically the formula for one-dimensional horizontal capillary flow in soils.

Irmay, S., The effect of air bubbles on the law of hydrostatic pressures in unsaturated soils, Commission des Eaux Souterraines, International Union of Geodesy and Geophysics, Ninth General Assembly, Brussels, 1951. (Unpublished).

The effect of air bubbles in the soil, the water phase being continuous, is found quantitatively. When the pressure in the capillary zone above the water table descends below a critical value $p_o v_o/n$ (v_o is the initial volume of air in soil of porosity n at the atmospheric pressure p_o), the bubbles burst. Under isothermal conditions, the law of hydrostatic pressure is modified. The capillary rise is considerably higher and may exceed 10 meters. The solubility of air in water has a considerable effect and explains the observed hysteresis effect in the tension-water content curves on wetting and drying. When there is a geothermal gradient downwards, the pressure law is modified considerably, with a theoretical maximum at a certain depth.

Jacob, C.E., On the flow of water in an elastic artesian aquifer, *Trans. Amer. Geophys. Union*, vol. 21, pp. 574-586, 1940.

Derives the fundamental differential equation governing flow of water in an elastic artesian aquifer assuming an infinite areal extent and a uniform initial distribution of head. Application is shown of determination of coefficients of storage and transmissibility by graphical method using pumping test data (Theis method). Coefficient of storage from tidal efficiency is also shown, based on an increased uniform load decreasing the volume of the aquifer and assuming no leakage of water from the aquifer. The amount of "storage" derived from compression of the aquifer and adjacent clay beds is found to vary from 3.4 to 17.6 times as great as the amount derived from expansion of the water. Mentions that the rate of transmission of artesian pressure is equal to the velocity of sound in the fluid, but that the apparent measured rates are a direct function of the accuracy of the methods employed.

Jacob, C.E., Radial flow in a leaky artesian aquifer, *Trans. Amer. Geophys. Union*, vol. 27, no. 2, pp. 198-205, 1946.

A partial differential equation is set up for radial flow in an elastic aquifer into which there is vertical leakage in proportion to the drawdown. This equation is integrated to obtain two steady state solutions, one for the case of a well in an infinite aquifer, and the other for the case where the head is maintained constant along an outer boundary concentric with the well. In the second case, the solution of the non-steady state is also obtained for flow towards a well discharging at a steady rate, the initial state being one of uniform head distribution. A table and curves are given for one set of assumed values of three of the parameters of the system.

Jacob, C.E., Drawdown test to determine effective radius of artesian well, *Trans. Amer. Soc, Civil Engrs.*, vol. 112, pp. 1047-1070, 1947.

Drawdown in an artesian well that is pumped has two components: The first, arising from the "resistance" of the formation, is proportional to the discharge; and the second, termed "well loss" and representing the loss of head that accompanies the flow through the screen and upward inside the casing to the pump intake, is proportional approximately to the square of the discharge. The resistance of an extensive artesian bed increases with time as the ever-widening area of influence of the well expands. Consequently, the specific capacity of the well, which is discharge per unit drawdown, decreases both with time and discharge. The multiple-step drawdown test outlined permits the determination of the well loss and of the "effective radius" (defined as the distance, measured radially from the axis of the well, at which the theoretical drawdown based on a logarithmic head distribution equals the actual drawdown just outside the screen) of the well. The trend of drawdown is observed in the pumped well and in one or more nearby observation wells as the discharge is increased in stepwise fashion. A simple graphical procedure gives the permeability and the compressibility of the bed. From these several factors it is possible to predict the pumping level at any time for any given discharge.

Jacob, C.E., Flow of underground water, Engineering Hydraulics, (edited by H. Rouse), J. Wiley and Sons, N.Y., pp. 321-386, 1950.

Contains comprehensive discussions and rigorous derivations of basic theoretical material relating to ground water flow, together with illustrative problems and applications to specific flow fields. Topics covered include, experimental basis of Darcy's law, range of validity of Darcy's law, permeability coefficient, laboratory measurements and practical considerations, generalization of Darcy's law, significance of the ground-water velocity potential, continuity and compressibility, differential equations of steady flow, differential equation for unsteady flow, boundary conditions, flow in a sand of uniform thickness, flow in a sand of non-uniform thickness, radial flow to a well, a well in a uniform flow, flow between a well and recharge well, a well near a stream, method of images, specific capacity of a well in a steady state, multiple-well systems, tidal fluctuations in wells, radial flow to a well in an extensive aquifer, pumping-test methods for determining formation constants, characteristics of a typical well, intermittent operation of wells, method of images applied to unsteady flow, steady unidirectional flow, steady radial flow to a well, flow with constant uniform recharge, flow to a well between two streams, general theory of unconfined flow, and approximate treatment in limited cases of unsteady flow.

Leggette, R.M. and Taylor, G.H., The transmission of pressures in artesian aquifers, Trans. Amer. Geophys. Union, pp. 409-413, 1934.

Describes measurements using sensitive water-level recorders of fluctuations of water levels in artesian wells at varying distances from wells that are opened or closed. In Ogden Valley, Utah, changes in elevation were noted in 7 minutes at 2850 feet, 57 minutes at 3850 feet, and 3-13 hours at 2 miles. Exact times were not determined because of obscuring effects of atmospheric pressure changes. In Salt Lake Valley, Utah, similar results were observed although conditions were more complicated due to back pressure effects. Maximum distance of pressure change noted was 1.3 miles, and rate of pressure transmission was generally higher when manipulated wells were closed than when they were opened. Changes in pressure between wells of different depth were noted and are suggested to result from transmission through flexible confining beds. Concludes that smaller pressure changes take longer to reach observation wells than do larger changes, that transmission time will be longer between wells of different depths than between wells of equal depths, and that pressure transmission decreases both in rate and magnitude with distance.

Lewis, M.R., Flow of ground water as applied to drainage wells, Trans. Amer. Soc. of Civil Engineers, vol. 96, pp. 1194-1211, 1932.

Derives formulas for drawdown curves for three types of wells: artesian well with perforated casing extending through the water-bearing foundation; well in which the water table is in the water-bearing stratum and which penetrates its full depth; and open-bottom well, which just reaches the water-bearing stratum. Considers cases of definite and indefinite zones of influence for each of the well types.

Meinzer, O.E., Movements of ground water, Bull. Amer. Assoc. Petr. Geologists, vol. 20, no. 6, pp. 704-725, 1936.

Presents general discussion of Darcy's law, range of coefficients of permeability, methods of determining permeability, movement of water through subcapillary openings, direction of ground water movement, and pressure transmission in confined and unconfined aquifers.

Muskat, M., Two fluid systems in porous media; the encroachment of water into an oil sand, Physics, vol. 5, no. 9, pp. 250-264, 1934.

The problem of the encroachment of water into an oil sand is formulated as a new type of potential problem, namely, that of finding potential distributions in two regions of different "conductivities" which are separated by a surface, each point of which has velocity proportional to the vector gradient of the potential at the point. The cases of strictly linear, radial and spherical systems, in which the shapes of the interfaces are evident from symmetry requirements, are solved in detail and discussed graphically. Analytical and graphical solutions are presented for (1) systems with elliptical boundaries, (2) an infinite linear source driving fluid into an isolated sink. and (3) the history of a ring of fluid particles travelling from a source to a sink. The relation of the analytical results to the practical problems of the encroachment of water in sands are mentioned.

Muskat, M., The seepage of water through porous media under the action of gravity, Trans. Amer. Geophysical Union, Pt. 2, pp. 391-395, 1936.

Summarizes seepage flow based on the assumptions that the fluid moves towards the well-surface in vertical shells so that the horizontal component of the velocity is independent of depth and that the magnitude of this horizontal velocity is proportional to the slope of the free water surface. Derives the potential distribution for flow through a pervious gravity dam with vertical faces. Shows that theoretical and empirical fluid-head distributions are in agreement for radial flow into a well.

Muskat, M., The flow of homogeneous fluids through porous media, McGraw - Hill, N.Y., 1st. ed., 763 pp., 1937.

Summarizes fundamental theoretical knowledge of flow in porous media. Chapter headings include: Introduction; Darcy's Law and the measurement of the permeability of porous media; General hydrodynamic equations for the flow of fluids through porous media; Two-dimensional flow problems and potential-theory methods; Three-dimensional problems; Gravity flow systems; Systems of nonuniform permeability; Two-fluid systems; Multiple-well systems; The flow of compressible liquids through porous media; and The flow of gases through porous media.

Slichter, C.S., Theoretical investigation of the motion of ground waters, 19th. Annual Report, Part II, U.S. Geological Survey, pp. 295-384, 1899.

Presents a fundamental mathematical analysis of the flow of ground water. Primary topics covered include: laws of the rectilinear flow of ground water through a soil; general laws of the flow of ground waters; motion of ground water in horizontal planes; motion of ground water in vertical planes; and flow of artesian wells and their mutual interference. Concludes with a bibliography of 77 items on the motion of ground waters and related topics.

Slichter, C.S., The motion of underground waters, Water-Supply and Irrigation Paper No. 67, U.S. Geological Survey, 106 pp., 1902.

This paper treats the simpler and more general topics connected with the movement of ground water, whereas the author's paper in the 19th. Annual Report of the U.S.G.S. (which see) is concerned primarily with the theoretical and experimental aspects of the subject. Topics covered include origin, extent, and motion of ground water, surface zone of flow of ground water, deep zones of flow, recovery of underground water from the surface flows, and artesian and deep wells. Presents description and pictures of construction of subsurface dam built on Pacoima Creek, Los Angeles County, California in 1887-1890. The dam ranges from 25 to 50 feet in depth and is 600 feet in length. Construction consisted of digging a five-foot wide trench to bed rock, boarding with timbers, pouring a 2-foot wide concrete core wall and packing gravel around both sides of this wall. Although the dam was not completely water-tight, the use of collecting wells immediately upstream from the dam enabled use to be made of the entire ground water flow during drought periods.

Stokes, G.G., Mathematical proof of the identity of the stream lines obtained by means of a viscous film with those of a perfect fluid moving in two dimensions, Report of the 68th. Meeting of the British Association for the Advancement of Science, London, pp. 143-144, 1899.

Presents a simple mathematical proof to show that streamlines observed in an experiment (see Hele-Shaw, 1899) are identical with the theoretical streamlines of steady motion of a perfect liquid.

Theis, C.V., Equation for lines of flow in vicinity of discharging artesian well, Trans. Amer. Geophys. Union, pp. 317-320, 1932.

Derives the flow field for the case of a discharging well located in a homogeneous artesian aquifer of uniform thickness with a plane piezometric surface. Shows a diagram of the flow net and indicates applications, as the problem of a drainage well penetrating a confined water-bearing bed may be polluting the water drawn from the same bed by another well.

Theis, C.V., The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using ground water storage, Trans. Amer. Geophys. Union, Pt. II, pp. 519-524, 1935.

Derives the equation for the drawdown at any point around a well being pumped uniformly at anytime, based on the following assumptions: that the aquifer is homogeneous, that the aquifer is infinite in areal extent; that the well penetrates the entire thickness of the aquifer; that the coefficient of transmissibility is constant at all times and in all places; that the well pumped has an infinitesimal diameter; and that the aquifer is unconfined. For aquifers approximating these assumptions, shows that observed and computed drawdowns check very closely. Indicates also that the equation can be used to determine the coefficient of transmissibility in an aquifer by observing rate of recovery in a well. Theoretical curves show that there is no areal limit to drawdown but that rate and amount of drawdown decrease with distance from the pumped well.

Van Everdingen, A.F. and Hurst, W., The application of the Laplace transformation to flow problems in reservoirs, Trans. Amer. Inst. of Mining and Metallurgical Engineers, vol. 186, pp. 305-324, 1949.

The fundamental theory and data on the flow of fluids through permeable media in the unsteady state are presented. Solutions of the unsteady state flow equations are developed for the "constant terminal pressure case" and the "constant terminal rate case". In the first case the pressure at the terminal boundary is lowered by unity at zero time kept constant thereafter, and the cumulative amount of fluid flowing across the boundary is computed, as a function of the time. In the second case a unit rate of production is made to flow across the terminal boundary (from time zero onward) and the ensuing pressure drop is computed as a function of the time. Tables have been compiled from which curves can be constructed for the two cases, both in finite and infinite reservoirs. Most of the information presented is obtained by the help of the Laplace transformations, which simplify the more involved mathematical analyses.

Werner, P.W., Notes on flow-time effects in the great artesian aquifers of the earth, Trans. Amer. Geophys. Union, vol. 27, no. 5, pp. 687-708, 1946.

Presents comprehensive mathematical analysis of flow in artesian aquifers based on the following primary factors: unit weight of water, compressibility of water, volume elasticity of aquifer, porosity of aquifer, and permeability of aquifer. Assuming these factors as constant for any given aquifer, the differential equation of variable flow in a horizontal, homogeneous aquifer of uniform and comparatively small thickness is derived and employed in several applications or problems. Shows that for a sinusoidal fluctuation of pressure in the forebay area, the amplitude decreases to 1/535 of its original value after one oscillation. For the case of a sudden, permanent rise of the forebay

pressure, the increased pressure in the center of the aquifer obtains 50 per cent of its full value after some 15 years and reaches its full value only after some 75 years. Changes in the coefficient of permeability will cause large changes in these time-rates of pressure distribution. For the case of an instantaneous drawdown from a slit extending perpendicular to the gradient of the aquifer, assuming a constant infiltration rate in the forebay, the discharge will be very large at the beginning due to the release of water stored by compression, followed by a rapid decrease in discharge until a steady state condition of forebay recharge equal to discharge is reached in 100-200 years. A variation of the last case is also mentioned in which a slit is withdrawing water from the aquifer as before but a constant head is maintained in the forebay area. The discharge decreases rapidly and approaches a constant value after 50-100 years. Emphasizes the fact that initial water taken from an aquifer comes from compression storage and is not representative of long-time safe yield capacity. Finally, studies the case of a single well tapping an infinite aquifer having a horizontal piezometric surface. Derives the equation for the drawdown curve and presents a comparison between observed and calculated lowering of the piezometric level which shows good agreement. Mentions possibility of exhausting ground water in artesian aquifer of Great Basin in Australia because of depletion of water stored in compression.

APPENDIX C, PART III

REVIEW OF FORMULAS AND DERIVATIONS OF THE EQUILIBRIUM RATE OF SEAWARD FLOW IN A COASTAL AQUIFER WITH SEA-WATER INTRUSION

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SUMMARY

The problem of relating the seaward flow of fresh water in a confined aquifer to the length of intrusion of a saline wedge has been solved theoretically by both the University of California (Ref. 1) and the Los Angeles County Flood Control District (Ref. 2). Although their results are essentially identical, the two derivations are quite different. Consequently, the writer has undertaken this brief study of the problem for the California State Division of Water Resources for the purpose of clarifying the problem and the various possible solutions. Three separate methods of solution, giving identical results, are discussed herein.

* * * * * * * * *

1. Introduction

Because sea water is slightly denser than fresh water, it will tend to underride fresh water and intrude into aquifers which intersect the ocean floor. If a confined aquifer is entirely below sea level, and if there were no fresh water flow, the sea water would gradually displace all the fresh water in the aquifer. Without a continuing seaward flow of fresh water it is impossible to keep the sea water out, just as it is impossible to keep water in a long horizontal box with an open end.

However, with a steady seaward flow of fresh water a saline wedge forms which intrudes into the aquifer only a finite distance. Here again, it is instructive to think of the rough analogy with the open-ended box: with a constant flow of water towards the open end of the box, it is possible to maintain a certain amount of storage of water in the

box. The higher this rate of flow toward the end of the box, the greater will be the amount of water present in the box at any one time. So, similarly, in a confined aquifer, the larger the fresh water flow toward the ocean, the greater will be the portion of the aquifer occupied by the fresh water, and the less will be the sea-water intrusion.

Throughout this discussion, it is assumed that there is essentially no mixing between the fresh water and the sea water. Observation has shown this to be a reasonable approximation as interfaces between fresh water and sea water are quite well defined. Furthermore, this is to be expected because there is no opportunity for turbulent mixing in a fine-grained aquifer.

The problem at hand is the determination of the relationship between the following variables:

q = seaward rate of flow of fresh water, per unit width

L = length of intruded sea water wedge

M = thickness of pressure aquifer (assumed constant)

$S = \frac{w_s}{w}$, = ratio of unit weight of sea water to fresh water

K = coefficient of permeability of aquifer (assumed constant)

The idealized boundaries for this two-dimensional flow are shown in Figure 1.

In Appendix I of the University of California report (Reference 1) it is shown that

$$q = \frac{K(S-1)M^2}{2L}, \qquad (1)$$

Appendix E of the report of the Los Angeles County Flood Control District (Reference 2) gives the same result, but with a factor "n" in the

numerator; however, it is stated that "n" is extremely close to 1 for all practical cases.

Before discussing the derivation of this formula, it will be shown that this fresh-water flow with a sea-water interface can be made analogous to a fresh-water flow with an air interface.

2. Analogy of intrusion problem to free-surface flow with air-water interface.

First it should be noted that under equilibrium conditions the sea-water wedge is stationary, and there is no flow anywhere within this intruded wedge. Consequently, the flow problem deals entirely with the flow of fresh water with a boundary condition imposed by the interface between fresh water and sea water. Thus, when we speak of piezometric heads, henceforth we shall be referring only to fresh-water piezometric heads.

Darcy's Law for flow in a porous medium states that the velocity is proportional to the gradient of the piezometric head. In the face of other complexities of the problem, it is important not to forget that the ground water flow is governed by the distribution of piezometric head and not by the distribution of pore pressure.

Referring now to Figure 1, we can define exactly the flow problem that we are interested in. The aquifer is presumed to be horizontal, constant in thickness, and uniform in permeability. Point A is at the vertex of the sea-water wedge, and the length of intrusion EA is denoted by L. The curve, AD, represents the interface between the fresh water and sea water, and BC is the top surface of the aquifer. Both AD and BC must be streamlines. In the aquifer upstream from the line AB

... ... the flow ... uniform and horizontal; thus, AB is ... ... line. ... CD is the discharge face which is ... for simplicity.

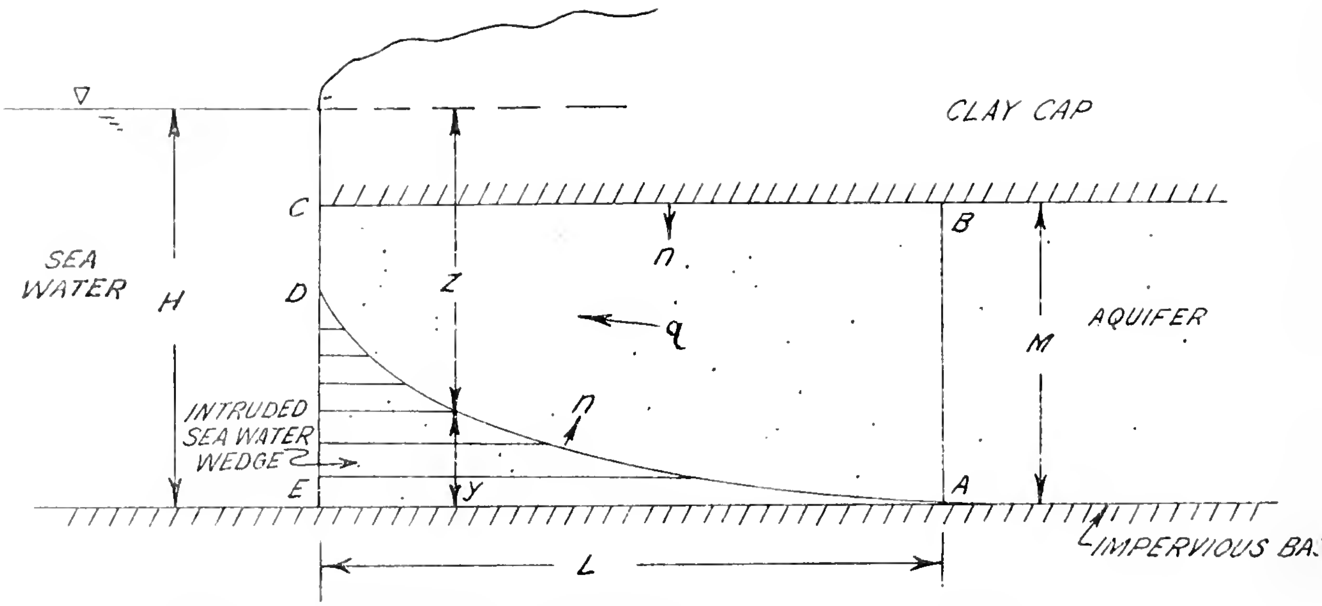

Figure 1. Idealized flow diagram for intrusion problem.

... evaluation of the piezometric head (or potential) along ... ries CD, AD, and AE may be conveniently done in the following ... using the base of the aquifer as the datum and letting z be the ... below sea level, the definition of piezometric head, h, gives

$$h = \frac{p}{w} + y$$

$$= \frac{p}{w} + (H-z)$$

... p is the pressure and y is the elevation above the datum and w is ... unit weight of fresh water. Anywhere within the static sea water ... the pressure $p = zw_s$, where w_s denotes the unit weight of sea water.

Consequently along AD or CD,

$$h = \frac{z w_s}{w} + H - z = (S-1)\, z + H \qquad (2)$$

If the datum is now moved to sea level for convenience, then we have, along AD or CD, simply

$$h = (S-1)\, z \qquad (3)$$

At point A, or hence along the entire line AB,

$$h = (S-1)\, H \qquad (4)$$

The boundary conditions for the intrusion problem can be summarized as follows:

AB : $h = (S-1)H$ (equipotential line)

BC : $\frac{\partial h}{\partial n} = 0$ (streamline) (i.e. no flow normal to BC)

CD : $h = (S-1)z$ (discharge face)

AD : $h = (S-1)z$ and $\frac{\partial h}{\partial n} = 0$ (streamline and interface)

In the above, $\frac{\partial h}{\partial n}$ is the rate of change at h in the direction normal to the boundary.

Now consider the analogous flow situation in Figure 2. The geometry of Figure 1 has been preserved, but the figure has simply been turned over and the sea water has been replaced by air. Points A', B', C', and D' of Figure 2 correspond to points A, B, C, and D respectively in Figure 1. In Figure 2 we have the familiar case where A' D' is the phreatic line, and C' D' is the discharge face. Now, in this case, the piezometric head on A', D' and C' D' is

$$h' = \frac{p'}{w} + z' = z' \qquad (5)$$

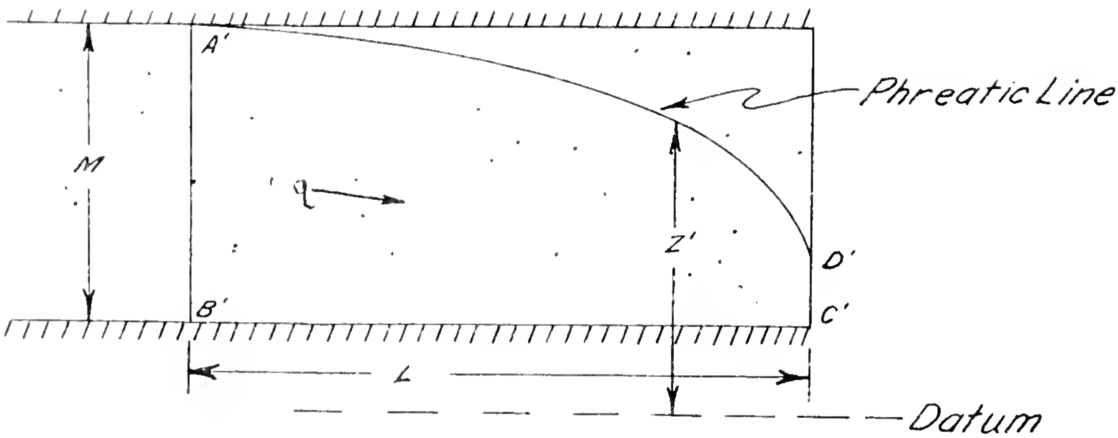

Figure 2. Analogous free surface flow

Now it may be noted that the flow problem of Figure 1 is exactly the same as the flow problem of Figure 2 except that all the values of the piezometric head are reduced by the constant factor $(G-1)$. Otherwise, the geometry and boundary conditions, as outlined above, are exactly the same. Consequently, all the values of the hydraulic gradient and the discharge are the same, except reduced by the factor $(G-1)$ in the case of the intrusion problem.

Henceforth, we shall talk about the analogous problem of Figure 2, because it is easier to visualize and identify with solutions already appearing in the literature.

3. Methods of Solution

Basically the solution of the flow problem involves solving the Laplace partial differential equation,

$$\frac{\partial^2 h}{x^2} + \frac{\partial^2 h}{y^2} = 0,$$

for the distribution of h in the region ABCD with the boundary conditions as summarized above. However, the rigorous solution of the Laplace equation is greatly complicated by the fact that the shape of the phreatic line is not given but is itself part of the solution. It can be done with difficulty by the method of hodographs (Reference 3, p. 300ff) or by numerical relaxation methods (Reference 4), but an approximate solution is adequate for the problem at hand.

Three approximate methods of solution will be discussed herein. When properly applied, all of them yield the same results, namely Eq. 1, which is believed to be within a per cent or two of the exact solution to the problem outlined above. In order of estimated reliability, the three approximate methods of solution are:

(a) Use of basic parabola

(b) Muskat's approximate potential theory

(c) Dupuit-Forchheimer theory

Method (b) is the basis of the derivation in the University of California report, whereas a combination of (a) and (c) is used by the Los Angeles County Flood Control District.

All of these approximate methods will be applied to the analogous problem of Figure 2 with the multiplying factor (S-1). We shall now consider them one at a time in the order listed.

4. Application of Basic Parabola.

A simple solution to the Laplace equation exists for a flow problem which is very closely related to that shown in Figure 2. This is the so-called Kozeny basic parabola solution in which the streamlines and potential lines are two families of confocal parabolas. (Reference 3, p. 325, or Reference 5). The boundary conditions for this solution, which are shown schematically in Figure 3, are slightly different from those given in Figure 2. First of all, the discharge face is horizontal instead of vertical; and secondly the flow is not bounded on the upstream side by a straight vertical potential line. However, if L is large compared with M, then it is readily seen that the error of using one of the potential lines, A" B" in place of the vertical line A' B' would be negligible. By the same token, it is apparent that it makes little difference whether the discharge face C" D" is horizontal or vertical as C' D' (Reference 6). (The horizontal discharge face, as used by the Los Angeles County Flood

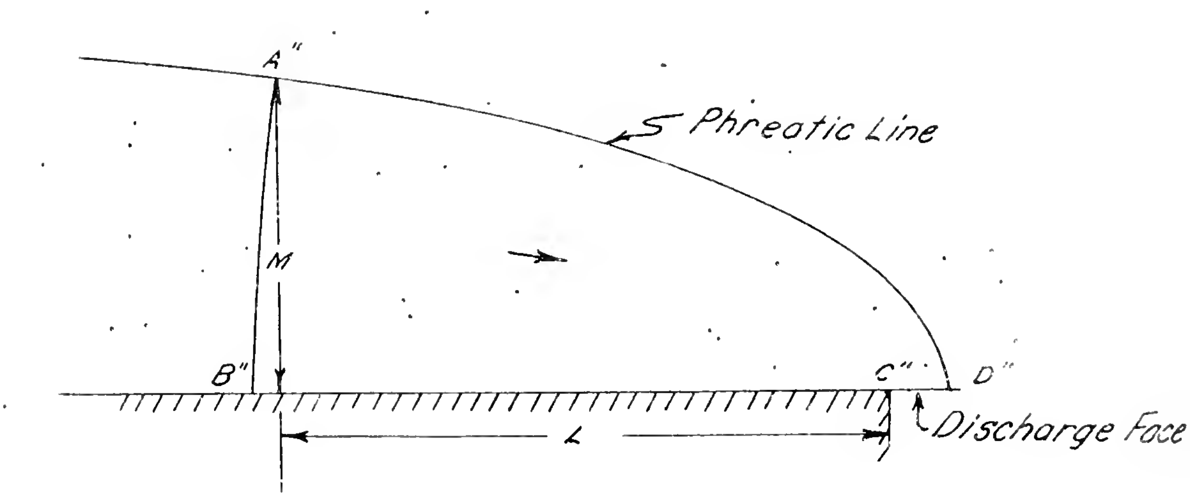

Figure 3. Flow diagram for Kozeny's basic parabola solution.

Control District, may be more realistic in the intrusion problem anyway). Therefore, it is believed that the solution found by application of the basic parabola will be very good, especially in a case where L is large compared with M as is usually the case.

The quantity of flow for the basic parabola solution is (Reference 5)

$$q = 2K(C''D'') \tag{6}$$

By the geometric properties of a parabola passing through A" and having its focus at C",

$$C''\,D'' = \tfrac{1}{2}\left(\sqrt{L^2 + M^2} - L\right)$$

Hence

$$q = K\left(\sqrt{L^2 + M^2} - L\right) \tag{7}$$

$$= KL\left(\sqrt{1 + \frac{M^2}{L^2}} - 1\right)$$

Now, assuming that $\frac{M}{L} << 1$, the radical may be replaced approximately by the first two terms of its binomial expansion. Thus

$$q = KL\left(1 + \tfrac{1}{2}\,\frac{M^2}{L^2} \; . \; . \; . -1\right)$$

or

$$q = \frac{KM^2}{2L} \tag{8}$$

Now, converting back to the intrusion problem, we need only insert the factor (S-1) thereby obtaining

$$q = \frac{K(S-1)M^2}{2L}, \tag{9}$$

which is identical to Eq. 1. It is believed that this solution should be correct within a very few per cent whenever $\frac{M}{L} < 0.02$.

5. Muskat's approximate potential theory.

On pages 377-380 of Ref. 3, Muskat solves a more general case of the problem outlined in Fig. 2 by his approximate potential theory. The solution for the distribution of piezometric head, h, given in the University of California report (Ref. 1) on page 46, Eq. I-2, is obtained exactly from Eq. 2 on page 379 in Muskat by taking the tailwater elevation $h_W = 0$ and changing nomenclature as required. Of course, the density factory (S-1) is also inserted to adopt the flow to the intrusion problem. The writer has also checked Eq. I-2 of the University of California report and Muskat's Eq. 2, p. 379, by direct derivation from Muskat's basic principle.

The calculation of the flux from the distribution of the piezometric head (Eq. I-2, Ref. 1) is straightforward as indicated in the University of California report, or Muskat, and yields the results previously given as Eq. 1 or Eq. 9. In six cases where Muskat compared his approximate solutions with the exact solutions obtained by the method of hodographs (Ref. 3, p. 314) he found the error in the quantity of seepage to be less than 1% in all cases. Although his trial cases did not include one in which L M, it is believed that this theory would still be within a few per cent in predicting the quantity of seepage.

6. Dupuit-Forchheimer theory.

The Dupuit-Forchheimer theory is illustrated in the derivation of the Los Angeles County Flood Control District's report, Appendix E.

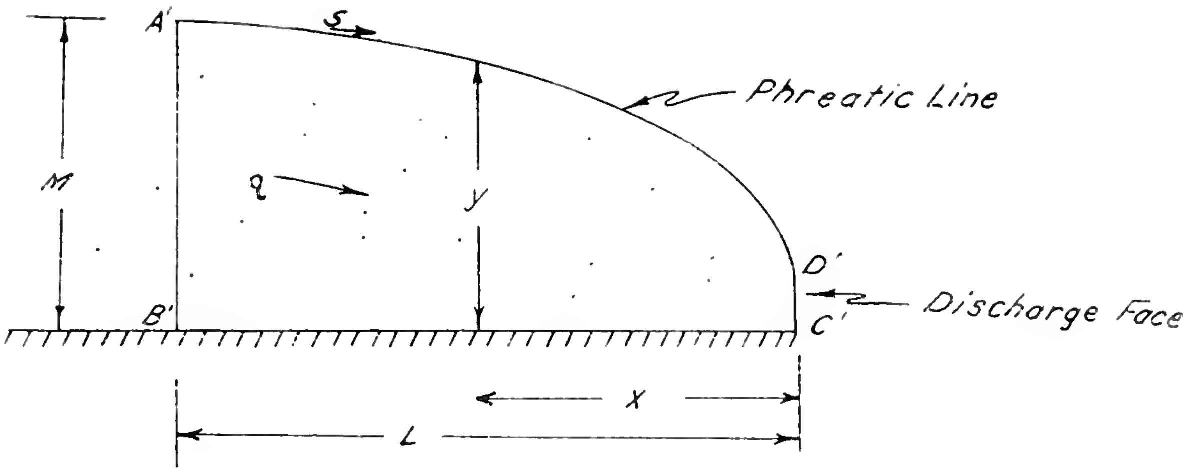

Figure 4. Coordinates Used in Applying the Dupuit-Forchheimer Theory.

Using the analogy with the free surface flow and coordinates as shown in Figure 4, the solution proceeds somewhat differently as follows.

It may be recalled first of all that Dupuit's assumptions are:

(a) the surface slope is small, so that the hydraulic gradient $\frac{dy}{ds} \approx \frac{dy}{dx}$.

(b) that the hydraulic gradient is constant on any vertical line and is equal to $\frac{dy}{dx}$.

(c) the velocity at all points is assumed to be parallel to the base.

Using these assumptions, and applying Darcy's Law, we may write for any vertical section

$$q = ky \frac{dy}{dx} \tag{10}$$

Integrating,

$$qx = k \frac{y^2}{2} + C$$

with the boundary condition y = M at x = L, we obtain

$$qL = K \frac{M^2}{2} + C, \quad \text{or} \quad C = qL - k \frac{M^2}{2}$$

consequently,

$$q(L - x) = \frac{k}{2} (M^2 - y^2) \tag{11}$$

now at $x = 0$, $y^2 << M^2$, so

$$q \approx \frac{kM^2}{2L} \tag{12}$$

and putting in the familiar factor (S-1) to make the analysis fit the intrusion problem, we have the result given in Eq. 1 again.

Muskat states that it is largely "fortuitous" that the Dupuit-Forchheimer theory yields such a good value for the total flux, especially in such a case as this where there is no tailwater, and hence large variations in depth of ground water flow.

The Flood Control District's report discusses a different way of handling the boundary condition at x = 0, by utilizing the basio parabola. The same result is obtained.

7. Conclusion

It has been shown how the theoretical intrusion problem can be solved readily by three different methods, leading to identical results. It is believed that the analogy between the intrusion problem and the ordinary ground water flow problem demonstrated in Section 2 is especially helpful in visualizing the problem and relating it to previous theoretical analyses in the literature.

Respectfully submitted,
/s/ Norman H. Brooks
NORMAN H. BROOKS
Civil Engineer

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